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Contents

Chapter **Page**

1 Introduction

1.1 Background 1
1.2 Purpose 1
1.3 References 1

2 Theory, Prediction, and Measurement of Odor and Corrosion 3

2.1 Introduction 3
2.2 Compounds Causing Odor and Corrosion 3
2.3 Mechanisms for the Generation of Hydrogen Sulfides 8
2.4 Mechanisms of Corrosion 17
2.5 Predicting Sulfide Buildup and Corrosion in Sewers 20
2.6 Approach to Investigating Odor and Corrosion 25
2.7 Measurement and Monitoring of Corrosion and Odor 29
2.8 Toxicity and Safety Practices 31
2.9 References 33

3 Odor and Corrosion Control in Existing Wastewater Collection Systems 35

3.1 Introduction 35
3.2 Improving the Oxygen Balance 35
3.3 Chemical Addition 53
3.4 Case Histories 61
3.5 References 66

4 Odor and Corrosion Control in Existing Wastewater Treatment Plants 69

4.1 Introduction 69
4.2 Sources of Odors in Wastewater Treatment Plants 69
4.3 Control of Odors in Existing Wastewater Treatment Plants 71
4.4 Corrosion in Wastewater Treatment Plants 94
4.5 Corrosion Control Techniques at Existing Wastewater Treatment Plants 95
4.6 Case Histories 96
4.7 References 100

5 Designing to Avoid Odor and Corrosion in New Wastewater Collection Systems 103

5.1 Introduction 103
5.2 Hydraulic Design 103
5.3 Ventilation of Sewers 110
5.4 Selection of Materials 112
5.5 References 116

Contents (continued)

Chapter **Page**

6 Designing to Avoid Odor and Corrosion in New Wastewater Treatment Facilities 117

- 6.1 Introduction 117
- 6.2 Common Sites of Odor Generation 118
- 6.3 General Design Considerations for Avoiding Odor Generation and Release 119
- 6.4 Design Procedures for Specific Odor-Producing Unit Processes 121
- 6.5 General Design Considerations for Avoiding Corrosion 126
- 6.6 Paint and Coatings 127
- 6.7 Selection of Materials 130
- 6.8 References 132

Contents (continued)

Chapter **Page**

6 Designing to Avoid Odor and Corrosion in New Wastewater Treatment Facilities 117

- 6.1 Introduction 117
- 6.2 Common Sites of Odor Generation 118
- 6.3 General Design Considerations for Avoiding Odor Generation and Release 119
- 6.4 Design Procedures for Specific Odor-Producing Unit Processes 121
- 6.5 General Design Considerations for Avoiding Corrosion 126
- 6.6 Paint and Coatings 127
- 6.7 Selection of Materials 130
- 6.8 References 132

Figures

Number **Page**

2-1 The Sulfur Cycle 5
2-2 Effect of pH on Hydrogen Sulfide Equilibrium 6
2-3 Proportions of H_2S and HS^- in Dissolved Sulfide 7
2-4 Relationship of Dissolved to Total Sulfide Concentration in the Sacramento, CA Central Trunk Sewers 8
2-5 Processes Occurring in Sewers with Sufficient Oxygen to Prevent Sulfide from Entering the Stream 9
2-6 Processes Occurring in Sewers Under Sulfide Buildup Conditions .. 10
2-7 Reaeration Rates in Sewers Flowing Half Full 12
2-8 Relative Reaeration Rates in a Sewer 12
2-9 Equilibrium Concentration of H_2S in Air 14
2-10 Effect of Velocity and Pipe Size on Sulfide Flux to Pipe Wall Under Specified Conditions 16
2-11 Factor to Apply to ϕ_{sw} from Figure 2-10 to Calculate ϕ_{sw} for Other than Half-Full Flow 16
2-12 Chemical Reactions in the Corrosion of Iron 18
2-13 Distribution of Corrosion in a Sewer 18
2-14 Specific Sulfide Flux Coefficients from Filled-Pipe Data 22
2-15 Observed and Predicted Corrosion Penetration, Sacramento, CA Central Trunk Sewer 25
2-16 Composite H_2S Mass Profile Entering Plant No. 2, Orange Co., CA 27
2-17 Typical Concentration-Intensity Relationship for Odors 30
2-18 Hydrogen Sulfide Toxicity Spectrum 31

3-1 Typical System for Injecting Air into Force Main 36
3-2 Typical Venturi Aspirator 37
3-3 Application of Venturi Aspirator for Sulfide Control in Wet Well 37
3-4 Air Lift Aerator Installed at Jefferson Parish, LA 38
3-5 Typical U-Tube Installation 38
3-6 Pressure Tank Dissolution System 39
3-7 Example of Determination of Oxygen Uptake Rate 41
3-8 Example of Variation in Oxygen Uptake Rate with Time 41
3-9 Example Ratio of Cumulative Oxygen Demand to Initial Oxygen Uptake vs. Time 42
3-10 Head Loss in Downleg of U-Tubes at Jefferson Parish, LA and Sacramento, CA 44
3-11 Typical Compressed-Air U-Tube Parametric Designs 45
3-12 Typical Aspirated-Air U-Tube Parametric Designs 45
3-13 Saturation Concentration of Oxygen in Water at Different Pressures 46
3-14 Typical Direct Oxygen Injection System 46
3-15 Multiple U-Tube Dissolution System 48
3-16 Typical Sidestream Dissolution System 48
3-17 System for Oxygen Injection into a Hydraulic Fall 49
3-18 Typical Liquid Oxygen Station 49
3-19 Pressure Swing Adsorption—Basic Flow Diagram 50

Figures (continued)

Number **Page**
3-20 Dissolved Oxygen Increase in the Wastewater Across U-Tube vs. Head Loss Comparing Oxygen and Air Injection 51
3-21 Typical Design Curve for Oxygen U-Tube 52
3-22 Typical Chlorinator Installation 55
3-23 Typical Chlorine Diffusers for Gravity Sewers and Force Mains 55
3-24 Typical Reaction Profile of Hydrogen Peroxide in Wastewater 56
3-25 Typical Package H_2O_2 Dosing Installation 57
3-26 Typical Bulk H_2O_2 Storage and Injection System 58
3-27 Effects of Sodium Nitrate on Sulfide Generation in Bluff Cove Force Main; Los Angeles, CA 60
3-28 Impact of Shock Dosing with NaOH on Recovery of Sulfide Buildup Capacity; Los Angeles, CA 61
3-29 Sewer Service Area; Delta Diablo Sanitation District 7A, California 63
3-30 Wastewater Collection System with H_2O_2 Dosing Stations; Palm Beach County, FL 65
3-31 Average Wastewater Temperature, Flow, and H_2O_2 Used at Lift Station No. 229; Palm Beach County, FL 66
4-1 Sulfide Removal from Wastewater Using Potassium Permanganate 74
4-2 Results of Pilot Studies Using Potassium Permanganate for Removal of Dissolved Sulfides 74
4-3 Sulfide Removal from Wastewater Sludge Using Potassium Permanganate 74
4-4 Effectiveness of Potassium Permanganate Addition for Controlling H_2S Generation from Sludge Centrifugation 75
4-5 Effect of Nitrate Addition on H_2S Emissions from a Trickling Filter 75
4-6 Typical Wet Scrubber System 78
4-7 Odorous Gas Removal Efficiency as a Function of Chlorine Concentration at Top of Packed Tower 79
4-8 Typical Activated Carbon Filter for Odor Control 82
4-9 Performance of $KMnO_4$-Impregnated Activated Alumina Adsorbers on Hydrogen Sulfide 84
4-10 Typical Iron Oxide Filter for Odor Control 84
4-11 Performance of an Iron Oxide Filter for Odor Control 85
4-12 Typical Ozone System for Odor Control 86
4-13 Ozone Requirements for Various Air Flows and Ozone Dosages 87
4-14 Direct Flame Oxidation Systems 88
4-15 Catalytic Oxidizer with Heat Recovery 88
4-16 Flow Diagram for a Direct Flame Oxidation System 89
4-17 Soil/Compost Filter for Odor Control 90
5-1 Sulfide Occurrence in Small Sewers 103
5-2 Minimum Scour Velocity Based on Boundary Shear Stress 104
5-3 Flow-Slope Relationships as Guides to Sulfide Forecasting 105
5-4 Alternative Sewer Grading Designs 106
5-5 Drop Manhole Design 106
5-6 Streamlined Junction 107
5-7 Pump Station with an Air Bypass 109
5-8 Air-Lift Pump Station 109
5-9 Forced Draft Ventilation for Odor Control; Austin, TX 112
5-10 T-Lock PVC Liner for Concrete Pipe 115
6-1 Typical Design of a Septage Receiving Station 125

Tables

Number **Page**

2-1 Odorous Sulfur Compounds in Wastewater 4
2-2 Logarithmic Ionization Constants (pK_1) for Hydrogen Sulfide 6
2-3 Solubility of H_2S in Water at a Pressure of 1 Standard Atmosphere 8
2-4 Physical and Chemical Properties of Hydrogen Sulfide 9
2-5 Expected Oxygen Adsorption in Wastewater Falls 12
2-6 Dissociation of Hydrogen Sulfide 15
2-7 Comparison of Predicted and Measured Total Sulfides in Sacramento Central Trunk 25
2-8 Comparison of Measured vs. Predicted Corrosion Penetration, Sacramento, CA Central Trunk Sewer 25
2-9 Interpretation of Results of Wastewater Survey 27
2-10 Hazardous Characteristics of Hydrogen Sulfide Gas 32

3-1 Selected Studies of Direct Injection of Compressed Air into Force Mains for Sulfide Control 36
3-2 Performance of Aspirated Air U-Tubes at Jefferson Parish, LA 38
3-3 Performance of Compressed Air U-Tubes at Port Arthur, TX 39
3-4 Performance of Compressed Air U-Tubes at Sacramento, CA 39
3-5 Performance of Pressure Tank Air Injection at Port Arthur, TX 40
3-6 Suggested Oxygen Reaction Rates 42
3-7 Typical Costs for Direct Compressed Air Injection into a Force Main for H_2S Control 45
3-8 Performance of Direct Oxygen Injection into Force Mains at Port Arthur, TX 47
3-9 Performance of Direct Oxygen Injection into Force Mains at Sacramento, CA 47
3-10 Performance of Oxygen U-Tube for Sulfide Control at Sacramento, CA 47
3-11 Performance of Multiple U-Tube Oxygen Injection 48
3-12 Typical Costs for Direct Oxygen Injection into Force Main for H_2S Control 53
3-13 Performance of Chlorination for Sulfide Control at Sacramento, CA 54
3-14 Effects of Chlorination on H_2S Concentrations in Force Main at Tampa, FL 54
3-15 Commercially Available Forms of Chlorine for Wastewater Applications 55
3-16 Typical Costs for Chlorine Injection for H_2S Control 56
3-17 Physical Properties of Hydrogen Peroxide 56
3-18 Performance of H_2O_2 for Sulfide Control in Wastewater Collection Systems 57
3-19 Typical Costs for Hydrogen Peroxide Injection for H_2S Control 58
3-20 Performance of $FeSO_4$ Addition for Sulfide Control 59
3-21 Typical Costs for $FeSO_4$ Injection for H_2S Control 59
3-22 DO Depletion in Raw Wastewater Westville Force Main, Woodbury, NJ 62

Tables (continued)

Number **Page**

3-23 Results of Air Injection into Westville Force Main, Woodbury, NJ ... 62
3-24 Costs for Sulfide Control in Westville Force Main Using H_2O_2 and H_2O_2 with Air Injection, Woodbury, NJ ... 63
3-25 Force Main Characteristics at Pure Oxygen Injection Points, Antioch, CA ... 63
3-26 Effectiveness of H_2O_2 for Sulfide Control at Palm Beach County, FL ... 66

4-1 Use of H_2O_2 to Control H_2S Odors at Baltimore, MD ... 73
4-2 Use of H_2O_2 for H_2S Odor Control in Sludge Handling System, Pittsburgh, PA ... 73
4-3 Performance of Nitrate Addition Compared with Other Measures for Sulfide Control in Activated Carbon Columns ... 76
4-4 Effectiveness of Hypochlorite Wet Scrubbers for Removal of Several Odorous Gases ... 78
4-5 Performance of Pilot-Scale Wet Scrubbers Using $KMnO_4$ and NaOH for H_2S Removal ... 79
4-6 Typical Costs for Wet Scrubbers for Odor Control ... 81
4-7 Physical Characteristics of Activated Carbon for Odor Control ... 81
4-8 Pilot Study on Sewer Odor Control Using Activated Carbon at Sacramento, CA ... 82
4-9 Typical Costs for Activated Carbon Adsorbers for Odor Control ... 83
4-10 Typical Costs for Ozone Systems for Odor Control ... 87
4-11 Agents that Adversely Affect Catalysts of the Platinum-Group Metals ... 88
4-12 Typical Costs for Thermal Incinerators for Odor Control ... 90
4-13 Typical Costs for Catalytic Incinerators ... 90
4-14 Performance of Compost Filter for H_2S Control at Moerewa, New Zealand ... 91
4-15 Estimated Costs of Compost Filters for Odor Control ... 92
4-16 Results of Wet Scrubber Performance Tests for H_2S Control, Santa Cruz County, CA ... 97
4-17 Design and Performance of Wet Scrubber/Activated Carbon Odor Control System at Tampa, FL ... 98

5-1 Oxygen Absorption in a Sewer With and Without a Drop ... 106

6-1 Potential for Odor Generation from Common Unit Processes in a Wastewater Treatment Plant ... 118
6-2 Matrix of Potential Odor-Producing Processes and Recommended Odor-Control Methods ... 122
6-3 Surface Preparations and Coatings for Various Environmental Exposures ... 128

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Chapter 1
Introduction

1.1 Background

Wastewater is known to the public for its potential to create odor nuisance. Sometimes it is the odors escaping from sewer manholes that cause complaints; more commonly, the odor source is a wastewater treatment facility. Yet there are wastewater treatment facilities that are free from this stigma, and techniques to prevent odor nuisances are available, to those committed to construct odor-free treatment works.

A major cause of odors in wastewater treatment systems is hydrogen sulfide (H_2S), a gas detectable in very low concentrations. H_2S is also notable for its toxicity and its ability to corrode various materials used in sewer and treatment plant construction.

In the last three decades, much research has been done on various aspects of the sulfide problem, and important contributions have been made by engineers in the United States, Australia (1) and South Africa (2).

Traditional sanitary sewer design practice has not fully acknowledged the importance of corrosion and odor control, as evidenced by the widespread occurrence of sulfide and odor control problems throughout the United States for sanitary sewers serving both small and large tributary areas. The 1984 EPA Needs Survey estimates the backlog cost of major sewer rehabilitation to be $3.2 billion (3). This cost is in addition to the costs for correcting infiltration/inflow problems and is for major structural repair or replacement of sanitary sewers, a significant part of which may be attributed to sulfide-induced deterioration. The same survey further estimates the construction costs for new collectors and interceptors through the year 2000 to be $38.8 billion. These cost estimates reflect the importance of adequately considering sulfide control in the design of new sanitary sewer systems.

Since publication of the Process Design Manual for Sulfide Control in Sanitary Sewerage Systems in 1974, substantial information on odors and corrosion in municipal sewerage systems has been reported. In addition, significant developments have evolved for the control of odors and corrosion in wastewater treatment plants. In particular, use of chemicals for odor and corrosion control has increased substantially. Inclusion of these advances is the primary reason for revising this manual. To further the understanding of odor and corrosion control in sewerage systems and treatment works, many case histories and examples have been added also.

1.2 Purpose

The need exists for a comprehensive design manual that brings together available information in a form convenient for those designing new systems or applying odor and corrosion control procedures in existing systems. This manual is intended to satisfy this need.

While sulfide control is now a well developed technology, continuing advances in basic knowledge and in control procedures are to be expected. Application of the art in its present state, however, as set forth in this manual, can overcome sulfide-producing tendencies in existing systems and help minimize future problems.

1.3 References

When an NTIS number is cited in a reference, that reference is available from:

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2. Stutterheim, N., and J.H.P. Van Aardt. *Corrosion of Concrete Sewers and Some Remedies.* South African Industrial Chemistry, No. 10, (1953).
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Chapter 2

Theory, Prediction, and Measurement of Odor and Corrosion

2.1 Introduction

Evaluation of existing or potential odor or corrosion problems requires knowledge of the types of compounds likely to cause such problems and the mechanisms for their formation in wastewater systems. Prediction of where such problems will occur in new systems is necessary so that designs can be tailored to minimize odor and corrosion.

Where odor and/or corrosion problems exist, a monitoring program should be developed to characterize the severity of the problems and to identify the sources of odor and corrosion. Such a program would involve careful sample collection and analysis, followed by interpretation of the data. Because collection of samples or inspection of sewers and wet wells can be hazardous, plant operators and sewer workers must be familiar with the potential dangers of confined spaces in contact with wastewater, and must strictly observe appropriate safety practices. Once sufficient data have been collected to fully characterize the problems, control options can be evaluated and a suitable control system implemented.

This chapter reviews odor and corrosion-causing compounds, and describes the mechanisms of sulfide generation. Methodologies for predicting sulfide generation and corrosion are also presented, as well as measurement and monitoring techniques to characterize new and existing wastewater collection and treatment systems.

2.2 Compounds Causing Odor and Corrosion

Odor-producing substances found in domestic wastewater and sludge are small, relatively volatile molecules with a molecular weight of 30 to 150 (1). Most of these substances result from the anaerobic decomposition of organic matter containing sulfur and nitrogen. Inorganic gases produced from domestic wastewater decomposition commonly include hydrogen sulfide, ammonia, carbon dioxide and methane. Of these gases, only hydrogen sulfide and ammonia are malodorous. Often, odor-producing substances include organic vapors such as indoles, skatoles, mercaptans and nitrogen-bearing organics.

H_2S is the most commonly known and prevalent odorous gas associated with domestic wastewater collection and treatment systems. It has a characteristic rotten egg odor, is extremely toxic, and is corrosive to metals such as iron, zinc, copper, lead and cadmium. H_2S is also a precursor to the formation of sulfuric acid, which corrodes lead-based paint, concrete, metals and other materials.

The conditions required for H_2S corrosion are (2):

1. Presence of dissolved sulfides in the wastewater.
2. Release of H_2S gas from the water phase to the gaseous phase.
3. Biological oxidation of H_2S to sulfuric acid above the wastewater surface in a pipe or basin
4. Acid attack on the moistened surfaces of cementitious or metallic surfaces exposed to the atmosphere.

The conditions leading to H_2S formation generally favor the production of other malodorous organic compounds. Investigations of the conditions favoring H_2S formation can also help to quantify the potential for odor generation from other compounds. Thus, solving H_2S odor problems can often solve other odor problems as well.

Many of the odors detected in wastewater collection and treatment systems result from the presence of sulfur-bearing compounds. A list of common malodorous sulfur-bearing compounds is shown in Table 2-1. The lower the molecular weight of a compound, the higher the volatility and potential for emission to the atmosphere. Substances of high molecular weight are usually not perceptibly odorous and are neither volatile nor soluble. Mercaptans are commonly found in wastewater and are analogous to alcohols with a substitution of sulfur for oxygen in the [OH] radical. Mercaptans are a reduced form of organic sulfur compounds. They are malodorous and can contribute to odor problems due to their extremely low threshold odor numbers (concentration below which a substance is no longer detectable by the human nose), as shown in Table 2-1 (3).

2.2.1 Sources of Sulfur in Domestic Wastewater

Sulfur is present in human excreta and sulfates are

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Table 2-1. Odorous Sulfur Compounds in Wastewater (3)

Substance	Formula	Characteristic Odor	Odor Threshold ppm	Molecular Weight
Allyl Mercaptan	$CH_2{=}CH\text{-}CH_2\text{-}SH$	Strong garlic-coffee	0.00005	74.15
Amyl Mercaptan	$CH_3\text{-}(CH_2)_3\text{-}CH_2\text{-}SH$	Unpleasant-putrid	0.0003	104.22
Benzyl Mercaptan	$C_6H_5CH_2\text{-}SH$	Unpleasant-strong	0.00019	124.21
Crotyl Mercaptan	$CH_3\text{-}CH{=}CH\text{-}CH_2\text{-}SH$	Skunk-like	0.000029	90.19
Dimethyl Sulfide	$CH_3\text{-}S\text{-}CH_3$	Decayed vegetables	0.0001	62.13
Ethyl Mercaptan	$CH_3CH_2\text{-}SH$	Decayed cabbage	0.00019	62.10
Hydrogen Sulfide	H_2S	Rotten eggs	0.00047	34.10
Methyl Mercaptan	CH_3SH	Decayed cabbage	0.0011	48.10
Propyl Mercaptan	$CH_3\text{-}CH_2\text{-}CH_2\text{-}SH$	Unpleasant	0.000075	76.16
Sulfur Dioxide	SO_2	Pungent, irritating	0.009	64.07
Tert-butyl Mercaptan	$(CH_3)_3C\text{-}SH$	Skunk, unpleasant	0.00008	90.10
Thiocresol	$CH_3\text{-}C_6H_4\text{-}SH$	Skunk, rancid	0.000062	124.21
Thiophenol	C_6H_5SH	Putrid, garlic-like	0.000062	110.18

found in most water supplies. Sufficient sulfur is normally available in domestic wastewater in the form of inorganic sulfates and sulfides such as mercaptans, thioethers, and disulfides for the production of odorous gases by anaerobic and facultative bacteria.

The sulfate ion ($SO_4^=$) is one of the most universal anions occurring in natural waters. It occurs frequently in rainfall, particularly from air masses that have encountered metropolitan areas. Sulfate concentrations in wastewater can vary from only a few milligrams per liter (mg/l) to hundreds of milligrams per liter (4).

Organic sulfur compounds are present in excreta, with domestic wastewater containing 1 to 3 mg/l. All sulfur compounds in oxidized or reduced forms, organic or inorganic, represent a potential for sulfide production. Generally, for domestic wastewater, the main source of sulfide is sulfate. The sulfur cycle is shown in Figure 2-1 (5).

2.2.2 Nature of Sulfide Compounds

2.2.2.1 Sulfate Reduction

The serious odor and corrosion problems associated with the collection, handling and treatment of domestic wastewater are primarily the result of the reduction of sulfate to H_2S under anaerobic conditions, as shown by the following reactions:

$$SO_4^= + \text{organic matter} \xrightarrow[\text{bacteria}]{\text{anaerobic}} S^= + H_2O + CO_2 \qquad (2\text{-}1)$$

$$S^= + 2H^+ \longrightarrow H_2S \qquad (2\text{-}2)$$

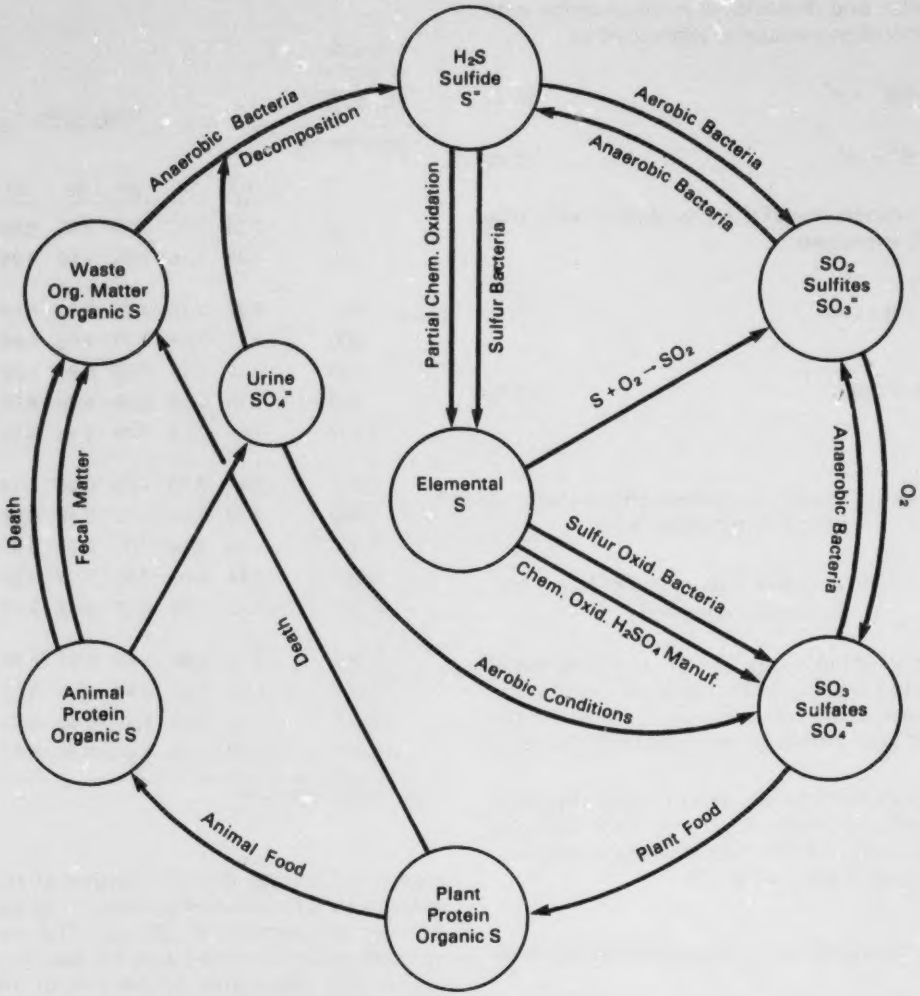
In the biochemical oxidation of organic matter, bacteria remove hydrogen atoms from the organic molecule and, in the process, gain energy. Through a series of biochemical reactions, the hydrogen atoms are transferred to a hydrogen acceptor. The hydrogen acceptor may be an inorganic or organic substance. Under aerobic conditions, free oxygen is the final acceptor for hydrogen, the oxygen being reduced to water. In the absence of free oxygen, combined oxygen may be used as a final acceptor of hydrogen.

The following reactions indicate the hydrogen acceptors and subsequent reduced products (6):

	Hydrogen Acceptor		Hydrogen Atoms Added	Reduced Product
(A)	O_2	+	$4\,H^+$	$2\,H_2O$
(B)	$2\,NO_3^-$	+	$12\,H^+$	$N_2 + 6\,H_2O$
(C)	$SO_4^=$	+	$10\,H^+$	$H_2S + 4\,H_2O$
(D)	Oxidized Organics	+	$x\,H^+$	Reduced Organics
(E)	CO_2	+	$8\,H^+$	$CH_4 + 2\,H_2O$

Reactions (A), (B) and (E) result in odorless products. Reaction (C) results in the malodorous H_2S, and reaction (D) often results in odorous products, such as mercaptans. The anaerobic reactions (B-E) occur only when oxygen is either absent or limited. Bacteria will utilize the hydrogen acceptors preferentially in the order given in the reaction list; i.e., oxygen first, nitrate second, sulfate third. However, not all microorganisms can use any hydrogen acceptor, as some are strictly aerobic bacteria. Others, such as obligate

Figure 2-1. **The sulfur cycle (5).**


H₂S
Sulfide
S⁼
Aerobic Bacteria
Anaerobic Bacteria
SO₂
Sulfites
SO₃⁼
Anaerobic Bacteria
Decomposition
Waste
Org. Matter
Organic S
Partial Chem. Oxidation
Sulfur Bacteria
S + O₂ → SO₂
Urine
SO₄⁼
Elemental
S
Anaerobic Bacteria
O₂
Death
Fecal Matter
Sulfur Oxid. Bacteria
Chem. Oxid. H₂SO₄ Manuf.
Death
Aerobic Conditions
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Protein
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Animal Food
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anaerobic bacteria, can use only combined forms of oxygen (NO_3^-, $SO_4^=$), while a large number of others are facultative and can use either free or combined oxygen as a hydrogen acceptor.

In the absence of dissolved oxygen (DO) and nitrates, sulfates serve as the hydrogen acceptor for biochemical oxidation by obligate anaerobic bacteria as expressed in reaction 2-1. The most important sulfate-reducing organism is the species *Desulfovibrio*. These bacteria are found both in the digestive tract of man and animals and in mud containing organic matter, and are normally present in domestic wastewater. The source of organic matter for this microorganism is quite restricted, and ammonia is the sole source of nitrogen.

2.2.2.2 Organic Compound Reductions

Proteins consist of amino acids, some of which contain sulfur. H_2S can be produced by the anaerobic decomposition of amino acids such as cysteine, cystine and methonine. This fermentation process is carried out by many species of proteolytic bacteria, including *Veillonella*, *Clostridia*, and *Proteus*(7). Although organic decomposition can contribute to H_2S production, sulfate reduction is the most significant mechanism for H_2S generation in wastewater.

2.2.2.3 Dissolved Sulfides

Molecular H_2S, formed from sulfate reduction, dissolves in water and dissociates in accordance with reversible ionization reactions, expressed as:

$$H_2S \longleftrightarrow HS^- + H^+ \qquad (2\text{-}3)$$

$$HS^- \longleftrightarrow S^= + H^+ \qquad (2\text{-}4)$$

The relative proportions of the species are related by the following expressions:

$$\log \frac{[HS^-]}{[H_2S]} = pH - pK_1 \qquad (2\text{-}5)$$

$$\log \frac{[S^=]}{[HS^-]} = pH - pK_2 \qquad (2\text{-}6)$$

where,

$[H_2S]$, $[HS^-]$, $[S^=]$ = molar concentrations of the respective constituents

pK_1, pK_2 = negative logarithms of the ionization constants

The distribution of the above species as a function of pH is shown in Figure 2-2 (5). It is apparent that the concentration of $S^=$ species is insignificant within the normal pH range of municipal wastewater (6.0 to 8.0).

Table 2-2 gives values for pK_1 as a function of specific electrical conductance (representing ionic strength) and temperature (8). For municipal wastewater, pK_1 = 7.0 is a reasonable approximation.

Figure 2-2. Effect of pH on hydrogen sulfide equilibrium (5).

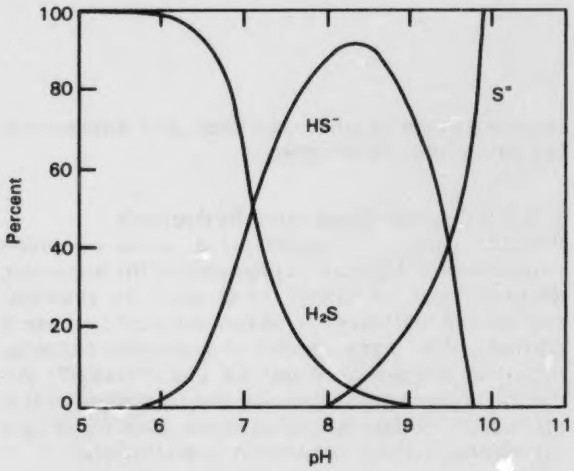


Table 2-2. Logarithmic Ionization Constants (pK_1) for Hydrogen Sulfide (8)

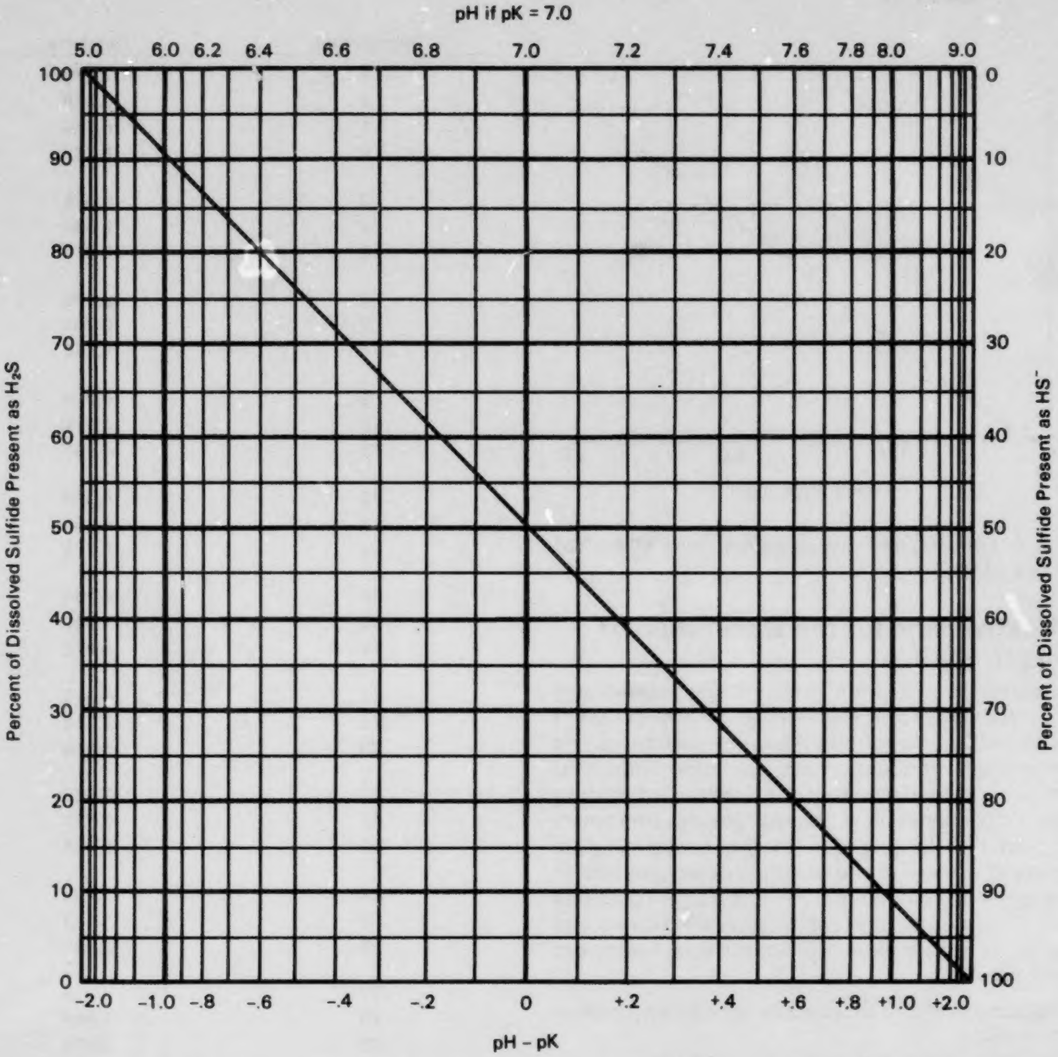
Specific Electrical Conductance @ 25°C micromhos/cm	Temperature, °C						
	10	15	20	25	30	35	40
0	7.24	7.17	7.10	7.03	6.96	6.89	6.82
25	7.23	7.16	7.09	7.02	6.95	6.88	6.81
100	7.22	7.15	7.08	7.01	6.94	6.87	6.80
200	7.21	7.14	7.07	7.00	6.93	6.86	6.79
400	7.20	7.13	7.06	6.99	6.92	6.85	6.78
700	7.19	7.12	7.05	6.98	6.91	6.84	6.77
1,200	7.18	7.11	7.04	6.97	6.90	6.83	6.76
2,000	7.17	7.10	7.03	6.96	6.89	6.82	6.75
3,000	7.16	7.09	7.02	6.95	6.88	6.81	6.74
4,000	7.15	7.08	7.01	6.94	6.87	6.80	6.73
5,200	7.14	7.07	7.00	6.93	6.86	6.79	6.72
7,200	7.13	7.06	6.99	6.92	6.85	6.78	6.71
10,000	7.12	7.05	6.98	6.91	6.84	6.77	6.70
14,000	7.11	7.04	6.97	6.90	6.83	6.76	6.69
22,000	7.10	7.03	6.96	6.89	6.82	6.75	6.68
50,000*	7.09	7.02	6.95	6.88	6.81	6.74	6.67

*Approximates sea water.

Figure 2-3 shows the distribution of H_2S and HS^- species as a function of pH-pK_1 or, assuming pK_1 = 7.0, as a function of pH (8). The relative H_2S concentration increases with decreasing pH. At a pH of 7.0, H_2S represents 50 percent of the dissolved sulfides present, while at a pH of 6.0, H_2S represents 90 percent of the dissolved sulfides. If part of the dissolved H_2S escapes to the atmosphere, the remaining dissolved sulfide will be divided between H_2S and HS^- in the same proportion as before because the equilibrium re-establishes itself almost instantly.

The distinction between the types of sulfide compounds is significant because only the H_2S can escape from solution and create odor and corrosion problems. It is important, therefore, to quantify the total and dissolved sulfides present and the pH of the wastewater. The amount of total sulfides occurring in the soluble form varies considerably in domestic wastewater, but most frequently appears to be 70 to 90 percent. The percentage of dissolved sulfides present varies with the pH of the wastewater and the amount of metals present.

Figure 2-3. **Proportions of H_2S and HS^- in dissolved sulfide (8).**



Total and dissolved sulfides from samples collected over a 15-year period from the Central Trunk Sewers in Sacramento, California are shown in Figure 2-4 (9). The dissolved sulfides in the Sacramento system are approximately 70 percent of the total sulfide. Based on a survey conducted on the River Oaks Plant collection system in May, 1982, Hillsborough County, Florida, reported dissolved sulfides averaging 88 percent of the total sulfides (10).

2.2.3 Physical and Chemical Properties of Hydrogen Sulfide

H_2S is a colorless gas that has a foul odor (rotten egg smell) and is slightly heavier than air. Human exposure to small amounts of H_2S in air can cause headaches, nausea, and eye irritation, and higher concentrations can cause paralysis of the respiratory system, which results in fainting and possible death. Concentrations of the gas of 0.2 percent are fatal to humans after exposure for a few minutes (11). The physiological effects of H_2S gas are discussed further in Section 2.8. H_2S gas is explosive at concentrations from 4.3 to 45.5 percent in air.

H_2S is moderately soluble in water, ranging between 3,000 and 4,000 mg/l at the normal temperatures found in wastewater. The solubility of H_2S decreases with increasing temperatures. Table 2-3 presents solubility data for H_2S at various temperatures (1).

Figure 2-4. **Relationship of dissolved to total sulfide concentration in the Sacramento, CA central trunk sewers (9).**

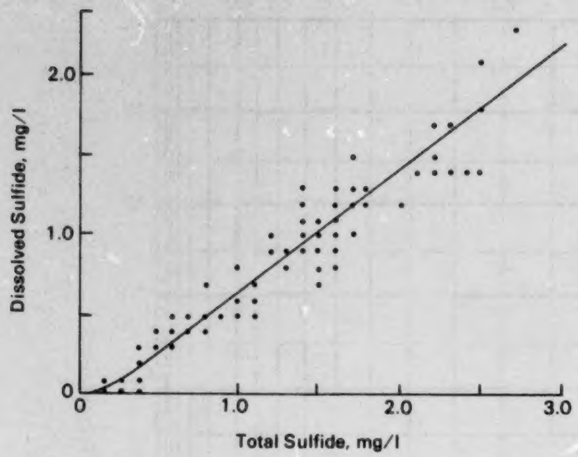


Table 2-4 summarizes the physical and chemical properties of H_2S.

2.3 Mechanisms for the Generation of Hydrogen Sulfide

The occurrence of sulfide in municipal wastewater results principally from the biochemical reduction of inorganic sulfur compounds. Although sulfide may be found in high concentrations in some industrial wastes or occasionally in ground water infiltrating sewers, its presence in municipal sewers and treatment plants is largely due to the bacteriological reduction of sulfate in the absence of oxygen and in the presence of organic matter. The sulfur reactions that are of particular interest in controlling odors and corrosion in wastewater collection and treatment systems are:

1. Reduction of sulfate or sulfur-containing matter to sulfide
2. Release of H_2S gas to the atmosphere
3. Oxidation of H_2S to sulfuric acid on the exposed walls of pipes and other structures

2.3.1 Sulfate Reduction

Many bacteria reduce sulfate to sulfide, including:

1. Assimilatory microbes—those that assimilate inorganic sulfur and reduce it to sulfide within their protoplasm
2. Proteolytic bacteria—several of which can hydrolyze proteins and amino acids under anaerobic conditions, resulting in the release of sulfides
3. Sulfate-reducing bacteria—specialized bacteria that use inorganic sulfate as the hydrogen acceptor in their energy cycle

Table 2-3. **Solubility of H_2S in Water at a Pressure of 1 Standard Atmosphere (1)**

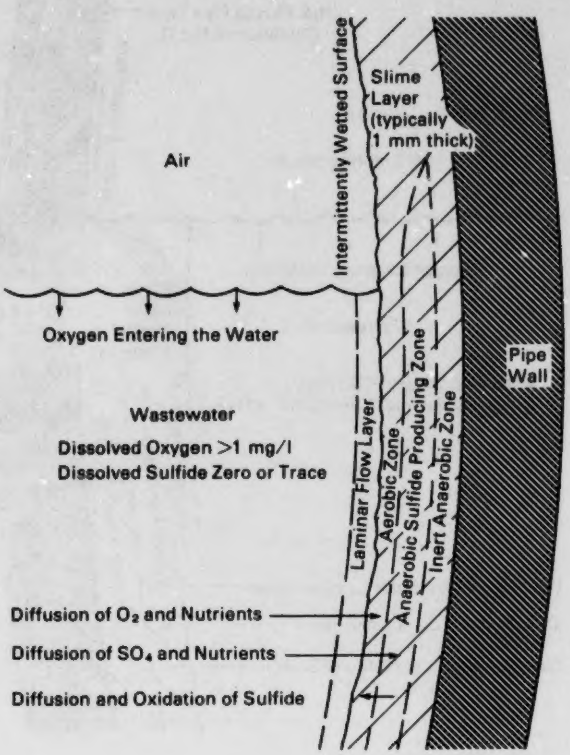
Temperature °C	Solubility mg/l as S
0	6,648
1	6,434
2	6,227
3	6,028
4	5,834
5	5,646
6	5,465
7	5,291
8	5,124
9	4,964
10	4,810
11	4,667
12	4,529
13	4,398
14	4,271
15	4,150
16	4,033
17	3,922
18	3,816
19	3,714
20	3,618
21	3,523
22	3,432
23	3,344
24	3,258
25	3,175
26	3,095
27	3,018
28	2,945
29	2,874
30	2,806
35	2,491
40	2,221

The sulfate-reducing bacteria, primarily *Desulfovibrio desulfuricans* (also called *Desulfatomaculum desulfuricans*), are the principal mechanism of sulfate reduction in municipal wasterwater collection and treatment systems. These are obligate anaerobes which utilize sulfate as the oxygen source (hydrogen acceptor) and various forms of organic matter as a food supply (hydrogen donor), including amino acids, carbohydrates, organic acids, etc. Using C to repre-

Table 2-4. Physical and Chemical Properties of Hydrogen Sulfide

Molecular Weight	34.08
Boiling Point, °C	-60.2
Melting Point, °C	-83.8 to 85.5
Vapor Pressure, -0.4°C	10 atm
25.5°C	20 atm
Specific Gravity (compared to air)	1.192
Auto Ignition Temperature, °C	250
Explosive range in air, percent	4.5 to 45.5

Figure 2-5. Processes occurring in sewers with sufficient oxygen to prevent sulfide from entering the stream (8).


Intermittently Wetted Surface
Slime Layer (typically 1 mm thick)
Air
Oxygen Entering the Water
Pipe Wall
Wastewater
Dissolved Oxygen >1 mg/l
Dissolved Sulfide Zero or Trace
Laminar Flow Layer
Aerobic Zone
Anaerobic Sulfide Producing Zone
Inert Anaerobic Zone
Diffusion of O₂ and Nutrients
Diffusion of SO₄ and Nutrients
Diffusion and Oxidation of Sulfide

sent organic matter, the reaction can be expressed as follows:

$$SO_4^{=} + 2\,C + H_2O \xrightarrow{\text{bacteria}} 2\,HCO_3^{-} + H_2S \qquad (2\text{-}7)$$

Sulfate, organic matter, and sulfate-reducing bacteria are present in virtually all wastewaters, yet sulfide generation does not always occur. Proper design and maintenance can often prevent odors and corrosion associated with sulfide generation.

Most of the sulfate reduction in sewers occurs in the biological slime layer on the pipe wall or in sludge and silt deposits on the pipe invert (8). These slimes are a matrix of filamentous organisms and gelatinous material (zoogleae) embedding smaller bacteria. Typically, slime layers are 0.3-1.0 mm (0.01-0.04 in) thick, although this varies depending on velocity and abrasive content of the wastewater and other environmental conditions.

Oxygen in wastewater diffuses into the slime layer. The extent of diffusion into the film is limited by the rapid oxygen utilization by aerobic bacteria near the surface of the layer. Beneath the aerobic zone, anaerobic conditions may prevail, providing conditions for sulfate reduction to occur. Closest to the pipe wall, the slime layer is anaerobic but largely inactive due to limited diffusion of nutrients. As long as an aerobic zone is present in the slime layer, sulfide diffusing out of the anaerobic zone will be oxidized and will not enter the wastewater stream. This condition is shown in Figure 2-5.

The relative thickness of the aerobic and anaerobic zones is determined by oxygen supply. The depth to which oxygen will penetrate is dependent on the oxygen concentration in the wastewater as well as on temperature and concentration of organic matter. If the oxygen concentration in the stream approaches zero (i.e., < 0.1 mg/l) insufficient oxygen will be present in the slime to oxidize all of the sulfide diffusing out of the anaerobic zone, and sulfide will enter the stream. This condition is shown in Figure 2-6.

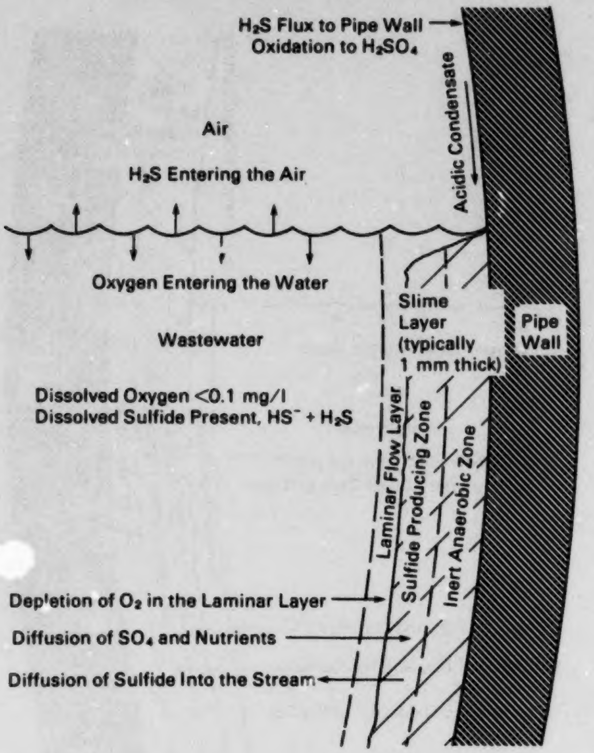
If the stream is stationary or moving slowly, local anaerobic conditions may occur near the pipe wall and some sulfide may escape, even though the DO concentration in the bulk liquid may be several milligrams per liter. However, completely anaerobic conditions must be approached for all of the sulfide to pass into the wastewater.

The two other reactions previously mentioned, the escape of H_2S into the sewer atmosphere and the oxidation of H_2S to sulfuric acid, are equally important in odor control.

2.3.2 Rate of Sulfide Production

The rate at which sulfide is produced by the slime layer depends on the following environmental conditions:

Figure 2-6. **Processes occurring in sewers under sulfide buildup conditions (8).**


H_2S Flux to Pipe Wall
Oxidation to H_2SO_4
Acidic Condensate
Air
H_2S Entering the Air
Oxygen Entering the Water
Wastewater
Slime Layer (typically 1 mm thick)
Pipe Wall
Dissolved Oxygen <0.1 mg/l
Dissolved Sulfide Present, $HS^- + H_2S$
Laminar Flow Layer
Sulfide Producing Zone
Inert Anaerobic Zone
Depletion of O_2 in the Laminar Layer
Diffusion of SO_4 and Nutrients
Diffusion of Sulfide Into the Stream

- Concentrations of organic material and nutrients
- Sulfate concentration
- DO
- pH
- Temperature
- Stream velocity
- Surface area
- Detention time

2.3.2.1 Concentrations of Organic Material and Nutrients

Organic matter and nutrients must diffuse into the slime layer to be utilized by the sulfate-reducing bacteria. Little is known about specific nutrients utilized by the *Desulfovibrio* bacteria, but unless sufficient quantities are present, sulfide generation may be limited by their availability. Pomeroy has assumed that the concentration of these nutrients is proportional to the BOD in most municipal wastewaters, and that the rate of sulfide generation by the slimes is proportional to the BOD if excess sulfate is available (8). Thistlethwayte has postulated that the rate of sulfide production is proportional to both BOD and sulfate concentration, varying as $[BOD]^{0.8}$ and as $[SO_4^{=}]^{0.4}$ (2).

2.3.2.2 Sulfate Concentration

Sulfate and organic matter will be utilized by the sulfate-reducing bacteria in the ratio of approximately 2:1, depending on the nature of the organics. It is unlikely that both will diffuse into the sulfate-reducing zone in ideal proportions. If sulfate is abundant, the sulfide generation rate will be proportional to the organic matter and/or nutrient concentrations. If sulfate is limiting, sulfide generation will be proportional to the sulfate concentration. Pomeroy has determined that if sulfate is in excess, the rate of sulfide production will be relatively independent of sulfate concentration (12). It has been estimated that sulfate will cease to be a limiting factor in sulfide production in most wastewaters at concentrations of 20 to 100 mg/l (8).

Thistlethwayte has developed an empirical equation which links sulfide generation directly to sulfate concentration and which predicts increased sulfide generation with increasing sulfate concentration (2). However, this has been shown not to hold true when excess sulfate is available (12).

2.3.2.3 Dissolved Oxygen

The critical dissolved oxygen concentration in the wastewater below which sulfate reduction can occur is 0.1 to 1.0 mg/l. Above 1.0 mg/l, sulfate reduction will be eliminated because of increased redox potential and inhibition of *Desulfovibrio*. Presence of DO in the stream will also encourage growth of the aerobic portion of the slime layer, increasing the distance through which organic matter and sulfate must diffuse to reach the sulfate reducers. Any sulfide produced in the active anaerobic zone is likely to be oxidized as it passes back through the aerobic zone.

Early studies on sulfate reduction showed that H_2S would not be produced until the bacteria utilized all of the DO and reduced all of the nitrates (8). Oxygen is gained primarily through reaeration at the stream surface and through turbulence induced by junctions, drops, hydraulic jumps and other places where air and wastewater mix. Oxygen is lost through consumption by microorganisms present in the wastewater and in the slime layer during biochemical oxidation of organic matter. The rate of change of oxygen in the stream due to consumption in the slime layer can be estimated by:

$$R_a = 5.3\,[O_2](su)^{1/2}R^{-1} \qquad (2\text{-}8)$$

where,

R_a = loss of oxygen from the stream by reaction with the slime layer, mg/l-hr

5.3 = empirical coefficient

$[O_2]$= oxygen concentration, mg/l

s = slope of the energy grade line, m/l

u = stream velocity, m/s

R = hydraulic radius of the stream, m

The reaeration rate, R_f, is the rate of change of oxygen concentration due to absorption from the atmosphere, and is related to other factors by the following:

$$R_f = \frac{\phi_f}{d_m} = \frac{fD}{d_m} \tag{2-9}$$

where,

R_f = reaeration rate, mg/l-hr

ϕ_f = flux of oxygen per unit area, g/m^2-hr

d_m= mean hydraulic depth (defined as the cross sectional area of the stream divided by its surface width), m

f = exchange coefficient, m/hr

D = oxygen deficit, mg/l or g/m^3

It should be noted that the reaeration coefficient, K_2, used in the Streeter-Phelps equation and typically expressed in units of $days^{-1}$, is related to f and to R_f by the following equations:

$$K_2 = \frac{f}{d_m} \tag{2-10}$$

$$R_f = K_2 D \tag{2-11}$$

where,

K_2 = reaeration coefficient, hr^{-1}

Pomeroy and Parkhurst found that for wastewater flowing in a partially filled sewer, the exchange coefficient, f, can be predicted by the following equation (13):

$$f = 0.96(C_A)(T)(su)^{3/8} \tag{2-12}$$

where,

0.96= empirical coefficient applicable to wastewater streams

C_A = factor representing the effect of turbulence in creating additional air-water interface compared to slow moving streams

T = temperature coefficient, equal to unity at 20°C

C_A can be approximated by:

$$C_A = 1 + \left[\frac{0.17\,u^2}{g\,d_m}\right] \tag{2-13}$$

where,

0.17= empirical coefficient

g = gravitational constant = $9.8\ m/s^2$

Substituting the empirical equation for f (Equation 2-12) into the equation for R_f (Equation 2-9) yields:

$$R_f = 0.96\,C_A\,T\,(su)^{3/8}\,D(d_m)^{-1} \tag{2-14}$$

where,

R_f = reaeration rate, mg/l-hr

C_A = turbulence factor

T = temperature coefficient

s = slope of energy grade line, m/m

u = stream velocity, m/s

D = DO deficit, mg/l

d_m = mean hydraulic depth, m

Based on the preceding equations, Figure 2-7 was developed to show estimated reaeration rates for various size sewers flowing half full as a function of stream velocity. These curves assume a DO deficit of 7 mg/l, 1 atmosphere of pressure, an oxygen content in the sewer atmosphere of 20.9 percent, and a wastewater temperature of 20°C. In calculating the deficit, the solubility of oxygen must be corrected for pressure, oxygen content of the sewer atmosphere, and temperature. An increase in temperature increases K_2 and the rate of oxygen absorption, if the same DO deficit is assumed. However, the deficit would likely be less due to the decreased solubility of oxygen.

Figure 2-7. Reaeration rates in sewers flowing half full (8).

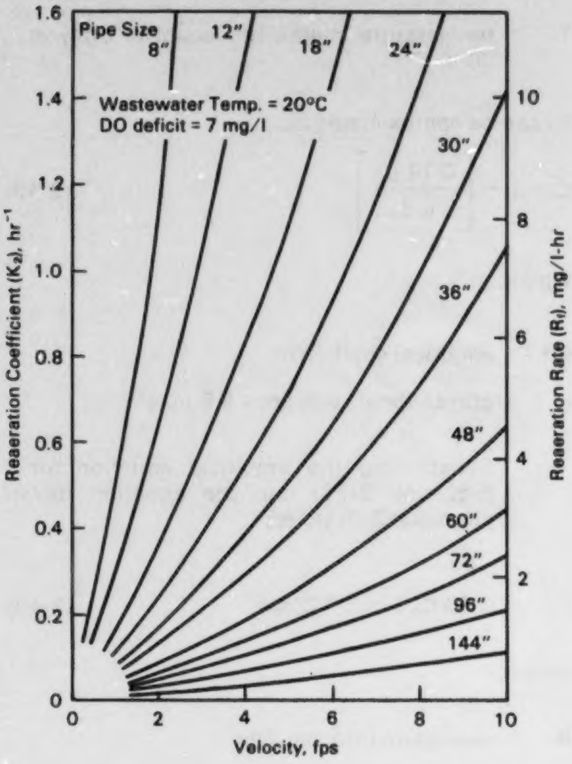


Figure 2-8 shows the relative values of K_2 and R_r in a sewer. These factors can be applied to the values obtained from Figure 2-7 to yield a value corrected for depth. However, if these two figures are used together in calculating reaeration rates for a sewer flowing other than half full, the input value used for velocity in entering the curve in Figure 2-7 must be the half-full velocity, not the actual velocity at other than half-full conditions.

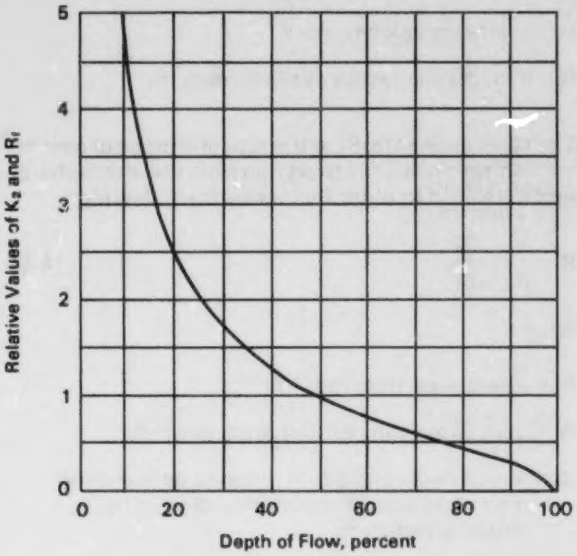
Oxygen may also be added through turbulence induced by junctions, drops, hydraulic jumps, etc. Pomeroy and Lofy found that in simple drops or falls, the oxygen concentration approaches saturation logarithmically with the height of the fall according to the following equation (14):

$$\ln \frac{D_1}{D_2} = K_H (H_1 - H_2) \qquad (2\text{-}15)$$

where,

D_1 and D_2 = oxygen deficits upstream and downstream from the drop, mg/l

Figure 2-8. Relative reaeration rates in a sewer (8).



K_H = waterfall reaeration coefficient, m^{-1}

H_1 and H_2 = elevations of the hydraulic grade line upstream and downstream from the jump, m

The average value of K_H for wastewater is approximately 0.41 m^{-1} (8). Using this value, Table 2-5 shows the percentage of the oxygen deficit that may be expected to be satisfied for various heights of fall.

Table 2-5. Expected Oxygen Absorption in Wastewater Falls (K_H = 0.41 m^{-1})

H_1-H_2	Oxygen Deficit Satisfied
ft	percent
1	12
2	22
3	31
4	39
5	46
6	53
8	63
10	71
15	87
20	92
30	98

In junctions where two or more sewers having different energy grade lines meet, one should use the difference between the average elevation of the energy grade lines entering the junction and the elevation of the outlet line for the combined flows to calculate H.

For large trunk sewers, reaeration rates may be quite slow due to the gentle slopes and large depth of flow in the pipe. However, a drop in a large sewer will have approximately the same effect on DO concentration as a drop in a small sewer. This follows from Equation 2-15, which shows reaeration from drop-induced turbulence to be independent of pipe size or flow rate.

Oxygen is consumed by microorganisms both within the stream and within the slime layer. Short-term oxygen consumption rates may vary according to distance travelled by the wastewater, and bear no correlation to standard BOD tests. Samples of wastewater collected near the upper ends of small sewers show relatively low oxygen consumption rates, on the order of 2 to 3 mg/l-hr. Oxygen consumption rates of the wastewater increase with distance travelled in the sewer due to sloughing of biologically active slimes from the pipe wall.

In larger trunk sewers, slopes are generally flatter and depths of flow greater. This results in a decrease in surface reaeration rates. At the same time, oxygen consumption rates are high (5 to 10 mg/l-hr) due to the distance travelled through the collector lines feeding the trunk. In trunks with diameters greater than 0.6 m (2.0 ft) flowing half full at a velocity of 0.6 m/s (2.0 ft/s), DO concentrations will approach zero if surface reaeration is the only mechanism of oxygen supply. Oxygen consumption rates will then be limited by oxygen supply.

In small sewers with low oxygen consumption rates and relatively high DO, a significant amount of oxygen consumption will occur at the slime layer. In larger sewers with high oxygen consumption rates in the wastewater, flatter slopes, and smaller surface area-to-volume ratios (greater hydraulic radii), oxygen consumption by the slime layer is insignificant.

When sulfide produced in the slime layer diffuses into a wastewater stream containing DO, sulfide may be oxidized chemically or biochemically. In typical municipal wastewaters, biological oxidation to thiosulfate is the prevalent mechanism, as follows:

$$2O_2 + 2HS^- \xrightarrow{\text{bacteria}} S_2O_3^= + H_2O \qquad (2\text{-}16)$$

Another mechanism, the oxidation of sulfide to elemental sulfur, does not occur to a great extent under normal conditions in municipal sewer systems. The rate of biochemical sulfide oxidation varies with the degree of biological activity, and may range from 1 to 2 mg/l-hr in fresh wastewater to 10 to 15 mg/l-hr for wastewater retained in the sewer for several hours. This rate is independent of sulfide and DO concentration in the wastewater as long as the concentrations of each are 1 mg/l or greater.

2.3.2.4 pH

The relative proportions of H_2S and HS^- are dependent on pH, which is of particular importance in assessing the potential for H_2S gas release into the sewer atmosphere, as discussed later in Section 2.3.3. The sulfate-reducing bacteria, however, are tolerant to changes in pH, being able to exist in a pH of 5.5 to 9.0 (8). The optimum pH for sulfate reducers is 7.5 to 8.0 (12).

2.3.2.5 Temperature

Temperature has a significant impact on the biological activity of the sulfate-reducers. It has been reported that the rate of sulfide production is increased 7 percent/°C up to 30°C (12). This is approximately equivalent to a doubling of the reaction rate for every 10°C increase in temperature. It has been theorized that, with increasing temperature, sulfide generation is further enhanced by the reduction in thickness of the laminar flow layer and by increase in nutrient supply to the sulfate reducers (2). However, as discussed in Section 2.3.2.6, this theory is inconsistent with other findings that reduction in laminar flow layer thickness by increased velocity did not affect the rate of sulfide production (8).

2.3.2.6 Stream Velocity

The rate of sulfide production is not directly altered by wastewater velocity (2)(12). Although a decrease in wastewater velocity is generally thought to increase the thickness of the slime layer, the thickness of the active sulfide-producing layer may remain unchanged, since the overall increase is possibly due to an increase in the inactive layer adjacent to the pipe wall (see Figure 2-6). Reduction in velocity would also tend to increase the thickness of the laminar flow layer, lengthening the distance through which nutrients must pass to reach the sulfate reducers. However, nutrients must also diffuse through the aerobic slime layer, and the overall transport rate is controlled by the layer of greater resistance.

Increased velocity will tend to reduce the thickness of the slime layer due to increased shear. However, the minimum thickness that will impair the rate of sulfide generation is not known. It has been shown that the rate of sulfide buildup in a pressure main flowing at 1.2 m/s (4.0 ft/s) was not significantly different from that in other mains flowing at lower velocities (8). It should be noted that when the velocity is less than scouring velocity, deposition of organic solids on the pipe invert may occur, and the deposited solids will serve as sites for sulfate reduction if anaerobic conditions develop.

In gravity sewers flowing less than full, higher velocities will result in turbulence-induced reaeration (see Equation 2-14). While this may not affect the rate of sulfide production from the anaerobic sulfide-producing zone in the slime layer, the DO caused by turbulence-induced reaeration will be used for the oxidation of sulfides in the aerobic zone of the slime layer or in the wastewater stream.

2.3.2.7 Surface Area

Factors which affect the pipe surface available for sulfate reduction in a wastewater collection system are flow rate, pipe diameter and energy gradient. These elements control depth of flow in the sewer. It follows that, as the depth increases, so does the surface available for development of slimes below the water level. Treatment plant structures such as wet wells, interprocess piping, and junctures that permit the accumulation of solids provide possible sites for sulfide generation.

2.3.2.8 Detention Time

As detention time in sewers, force mains, and non-aerated holding basins increases, the oxygen consumption increases, the oxidation-reduction potential (ORP) decreases, and organic matter becomes increasingly solubilized. These conditions favor the activity of the sulfate-reducing organisms. Thus, in the design of collection and treatment systems, minimizing detention time can limit the activity of the *Desulfovibrio* bacteria and thus the rate of sulfide production.

2.3.3 Hydrogen Sulfide Release

In enclosed vessels containing dissolved H_2S at equilibrium, the concentration of H_2S gas in the atmosphere will vary with the dissolved H_2S concentration according to Henry's Law:

$$K = \frac{p}{x} \qquad (2\text{-}17)$$

where,

K = Henry's Law constant, atm

p = partial pressure of the gas phase over the solution, atm

x = mole fraction of the dissolved gas in the liquid phase, dimensionless

Figure 2-9 shows the partial pressure of H_2S gas as a function of temperature for a range of concentrations of dissolved H_2S. At 1 atmosphere of pressure, the partial pressure in millionths of an atmosphere is equal to volumetric concentration in ppm. Thus, at 20°C, 3.0 mg/l of dissolved H_2S will be in equilibrium with approximately 780 ppm by volume of gaseous H_2S. Increasing the temperature decreases the solubility of the gas, and more will be present in the atmosphere.

2.3.3.1 Rate of Hydrogen Sulfide Gas Release

The extent and rate of H_2S gas release to the sewer atmosphere are controlled by the following factors:

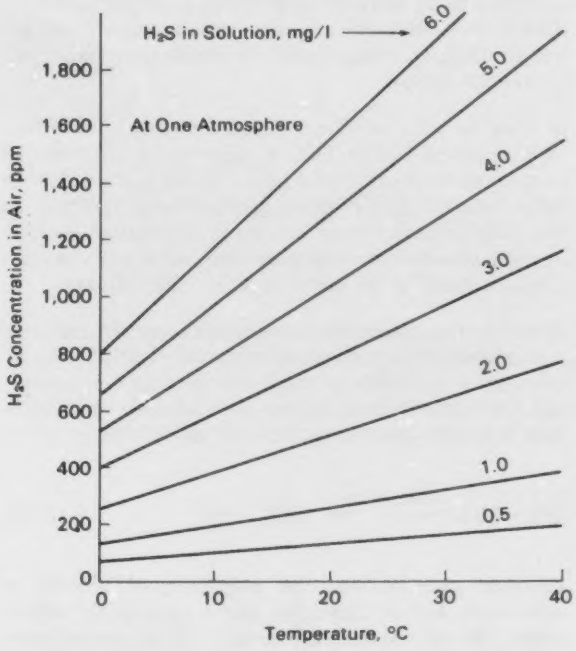
a. Dissolved Oxygen

Sulfides generated in the sulfate-reducing slime layer are likely to be oxidized in the aerobic layer or in the wastewater stream if the DO concentration is 1.0 mg/l or greater. If, however, the ORP and DO are low, some of the sulfide produced will diffuse into the stream.

b. pH

The dissociation of H_2S is dependent upon pH. The dissociation of HS^- to H^+ and $S^=$ is of minimal concern, as this is significant only at high pH values. The un-ionized H_2S is the only form of sulfide which can be

Figure 2-9. Equilibrium concentration of H_2S in air.



released to the sewer atmosphere. Lower pH values favor the un-ionized H_2S, and thus result in greater potential for release of the gas from the liquid. Table 2-6 illustrates the relative proportions of H_2S and HS^- as a function of pH, assuming a pK_1 of 7.0 and total dissolved sulfide content of 4.0 mg/l.

Table 2-6. **Dissociation of Hydrogen Sulfide (pK_1 = 7.0; [DS] = 4.0 mg/l)**

pH	$H_2S_{(aq)}$ mg/l	HS^- mg/l
6.0	3.6	0.4
6.5	3.0	1.0
7.0	2.0	2.0
7.5	1.0	3.0
8.0	0.4	3.6

c. Metal Concentration
Several metals typically found in municipal wastewater form insoluble metallic sulfides upon reaction with dissolved sulfide. Such metals include iron, zinc, copper, lead and cadmium. Given a typical range of pH for municipal wastewater, these metallic sulfides will continue to flow through the sewer without further reaction. The typical range of concentrations of insoluble metallic sulfides in domestic wastewater is 0.2 to 0.3 mg/l (8). However, industrial contributions of metal-bearing wastes may significantly increase the metals available for sulfide precipitation.

d. Velocity
Velocity is a factor in release of H_2S to the sewer atmosphere for two reasons:

1. Increased velocity induces turbulence, which increases the water surface area for gas transfer and increases H_2S release to the atmosphere.
2. Turbulence will likely increase the DO concentration in the stream due to surface reaeration, which may reduce H_2S gas release due to oxidation of sulfides (see Section 2.3.2.3).

e. Depth of Flow
Depth of flow in a sewer of a given size determines the cross-sectional area of flow and the water surface area, both of which affect gas transfer. These factors can be represented by the mean hydraulic depth, d_m, defined as the cross-sectional area of the stream divided by its surface width.

f. Temperature
Increased temperature increases the reaeration coefficient for a fixed oxygen deficit. However, increased temperature also decreases the oxygen deficit since solubility is reduced. The result is an offsetting of these two factors, the net effect of temperature on H_2S release being minimal (15).

2.3.3.2 Predicting Hydrogen Sulfide Gas Release
Pomeroy has developed an equation to predict the decline of sulfide in a stream due to loss of H_2S to the atmosphere:

$$R_{sf} = \frac{\phi_{sf}}{d_m} \qquad (2\text{-}18)$$

where,

R_{sf} = depletion of sulfide in the stream due to escape of H_2S, mg/l-hr

ϕ_{sf} = flux of H_2S from the stream surface, grams of sulfide per m^2-hr

d_m = mean hydraulic depth (defined as the cross-sectional area of the stream divided by its surface width), m

$$\phi_{sf} = 0.69\, C_A\, T\, (su)^{3/8}\, (1-q)\, j\, [DS] \qquad (2\text{-}19)$$

where,

C_A = factor representing the effect of turbulence in comparison to a slow stream (see Section 2.3.2.3)

T = temperature coefficient, equal to unity at 20°C

s = slope of the energy grade line of the stream, m/m

u = stream velocity, m/s

q = relative H_2S saturation in the air compared to equilibrium concentration (typically 2 to 20 percent), expressed as decimal fraction

j = proportion of dissolved sulfide present as H_2S (from Figure 2-3)

[DS] = dissolved sulfide concentration in the wastewater, mg/l

Under typical sewer conditions, excluding shallow, high velocity streams or any points of high turbulence, Equation 2-19 can be approximated by:

$$\phi_{sf} = 0.69\, (su)^{3/8}\, j\, [DS] \qquad (2\text{-}20)$$

If it is assumed that all of the H_2S escaping to the sewer atmosphere is oxidized on the pipe wall, the average sulfide flux to the wall can be calculated by multiplying the sulfide flux from the surface by the ratio of surface width to exposed wall perimeter:

$$\phi_{sw} = 0.69\,(su)^{3/8}\,j\,[DS]\,(b/P') \qquad (2\text{-}21)$$

where,

ϕ_{sw} = flux of H_2S to the pipe wall, g/m^2-hr

(b/P') = ratio of width of wastewater stream at surface to exposed perimeter of the pipe wall above the water surface

Figure 2-10 shows estimated sulfide flux to the pipe wall as a function of stream velocity for pipes flowing half full. The smaller pipe diameter yields higher flux rates due to increased turbulence. Figure 2-11 provides correction factors to be applied to the values from Figure 2-10 in order to calculate sulfide flux in pipes other than half full. Under these conditions, the value of velocity used to enter the curve in Figure 2-10 must be the half-full velocity, not the actual velocity at other than half-full conditions.

2.3.4 Hydrogen Sulfide Oxidation

The bacterial reduction of sulfate to sulfide, and the subsequent release of H_2S gas to the sewer atmosphere has been described. Corrosion of exposed concrete or metal surfaces occurs from the bacterial oxidation of H_2S to sulfuric acid under aerobic conditions. This is described by the following reaction:

$$H_2S + 2O_2 \xrightarrow{\text{bacteria}} H_2SO_4 \qquad (2\text{-}22)$$

This reaction is brought about by the action of *Thiobacillus* bacteria. Under acidic conditions, the principal bacteria are *Thiobacillus concretivorus*. These organisms are very tolerant to low pH, remaining active at sulfuric acid concentrations of 7 percent. They are autotrophic aerobes, which require a sulfur

Figure 2-10. **Effect of velocity and pipe size on sulfide flux to pipe wall under specified conditions (8).**

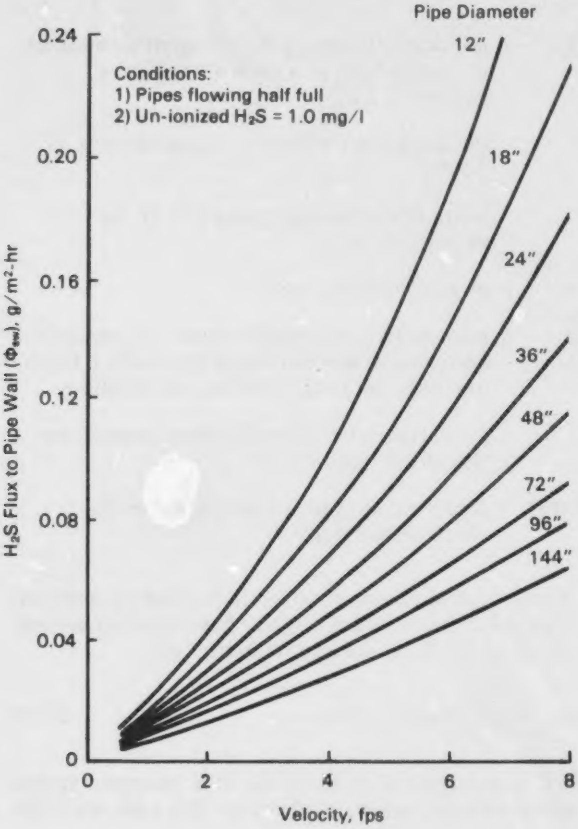
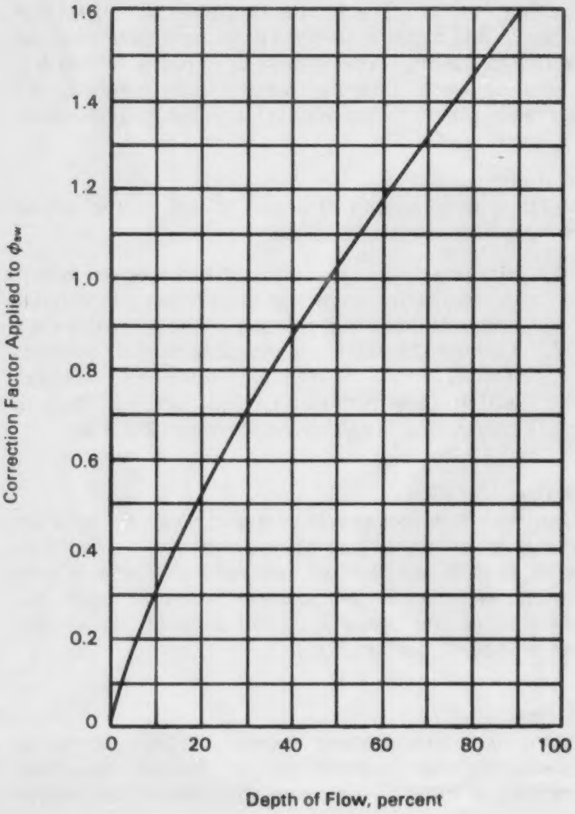


Figure 2-11. **Factor to apply to Φ_{sw} from Figure 2-10 to calculate Φ_{sw} for other than half-full flow (8).**



source (H_2S, elemental sulfur, or thiosulfate) for energy. Carbon dioxide provides the carbon source.

Moisture must be present on the exposed surfaces to support the bacterial metabolism necessary for the production of H_2SO_4. For new concrete, condensed moisture will be highly alkaline, with pH values ranging from 11.0 to 13.0. *Thiobacillus* are unable to survive under these conditions. Weathering of the concrete converts calcium hydroxide to calcuim carbonate, which then dissociates to bicarbonate, resulting in a decrease of surface pH. In addition, presence of carbon dioxide in the sewer atmosphere (approximately 1 percent by volume) drops the pH to about 7.4. *Thiobacillus thioparus* and similar bacteria will then establish themselves in the pH range of 5 to 9. These autotrophic bacteria oxidize the H_2S to thiosulfuric and polythionic acids, further reducing the pH of the condensate. *Thiobacillus thiooxidans* and *Thiobacillus concretivorus* then become established at $pH < 5$. These organisms are able to oxidize H_2S, elemental sulfur, thiosulfate and polythionates to H_2SO_4, a strong acid. This further reduces the pH, often to values below 2.0.

Sufficient moisture must be present on the pipe wall both for weathering to occur and for prevention of dessication of the sulfur bacteria. Thus, moisture is a primary factor affecting H_2S oxidation to sulfuric acid. In some cases, sewer ventilation is used to reduce the humidity in collection systems, thereby reducing the amount of condensate formed on the pipe wall, this may also increase the rate of surface reaeration

2.4 Mechanisms of Corrosion

Corrosion may be broadly defined as the destruction or deterioration of materials by the direct chemical or electrochemical reaction with their environment (16). Various types of corrosion are discussed in the following sections.

2.4.1 Direct Chemical Corrosion

2.4.1.1 Oxidation

Oxidation is the most familiar type of corrosion and is readily observable in the form of rust. Oxidation involves an exchange of electrons between the metal and free oxygen present in the environment. Examples of the reactions which occur during the oxidation of iron are shown in Figure 2-12. Metal oxides formed during oxidation are more electrochemically stable than the original metal. Thus, buildup of the oxide on a metal surface acts as insulation to reduce the rate of reaction. Oxides of chromium, aluminum, and nickel form a barrier of microscopic thickness that will effectively prevent further corrosion. However, some oxides, such as iron oxide, are not effective barriers, and the oxide itself may be porous or subject to chemical attack by such wastewater constituents as chlorides and sulfates (17).

2.4.1.2 Hydrogenation

Hydrogenation may occur when a metal is immersed in non-aerated water or non-oxidizing acid. Some of the water is reduced to its ions, H^+ and OH^-. Under high temperatures, pressures, and stress conditions, hydrogen penetrates the lattice structure of the metal and reacts with its internal structure. The internal changes which occur can cause loss of ductility (hydrogen embrittlement) and creation of internal pressures and splitting (hydrogen cracking). In more malleable metals, surface blistering results (17).

Separation of structural boundaries in a metal can be caused by increasing the temperature, roughening the surface, working the metal, or subjecting the metal to stress. These allow the hydrogen to penetrate the metal and attack exposed faces on interior surfaces. As the ions build up, they slowly join to form molecules of free hydrogen which are unable to escape and generate internal pressures (17).

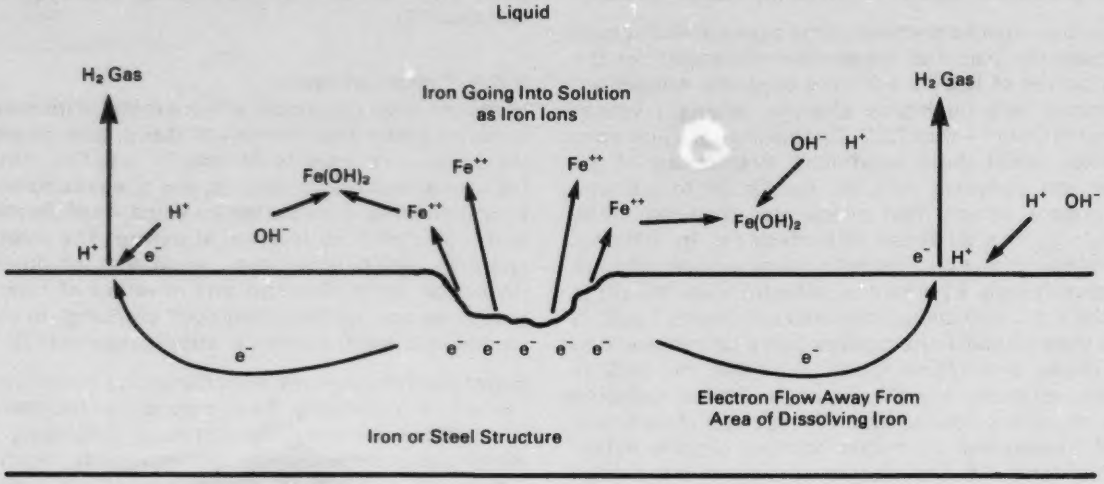
2.4.1.3 Other Direct Chemical Reactions

Corrosive chemicals such as chlorine, various acids and alkalies, and ferric chloride are commonly used in wastewater collection and treatment operations. In addition, byproducts of sewage treatment processes, such as sludge supernatant liquors, are very corrosive. Gases such as H_2S and SO_2 are also corrosive, both in the gaseous state and after reaction with water and oxygen to form sulfuric acid. In coastal areas, salt (sodium chloride) in water vapor can be very damaging. Dew can also be quite corrosive in industrial areas due to the absorption of corrosive gases from manufacturing operations (17).

H_2S gas can directly attack metallic components of wastewater systems such as steel tanks, structural members, gratings and walkways, and equipment (grit collectors, bar screens, conveyors, etc.). In addition, H_2S reacts directly with copper electrical components to form black copper sulfate, a poor conductor.

In collection systems, a major causative agent of corrosion is sulfuric acid formed from the oxidation of H_2S in the presence of moisture, the mechanism of which is described in Section 2.3. Corrosion of the pipe wall of a sewer is not uniform. This is due to several factors, including air currents, migration of sulfuric acid down the pipe wall, and exposure to water. The pipe wall is normally cooler than the wastewater, particularly during the summer. Air cooled by the walls moves downward along the walls, and is replaced by slightly warmer air that rises from the center of the stream surface. As a result,

Figure 2-12. **Chemical reactions in the corrosion of iron.**



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maximum rate of H_2S transfer to the pipe wall occurs at the crown. Acid formed on the pipe wall as a result of H_2S oxidation migrates down the wall toward the stream. Where corroded pipe has been removed for inspection, the effects of this migration can be seen as irregular vertical grooves in the pipe wall. Corrosion at the waterline is often severe. This is due to intermittent washing to the pipe wall in this zone, which cleans away the pasty decomposition products of concrete. This exposes new concrete, which is subject to rapid attack by the acid. Typical distribution of corrosion in the interior of a sewer is shown in Figure 2-13.

2.4.2 Bacteriological Corrosion

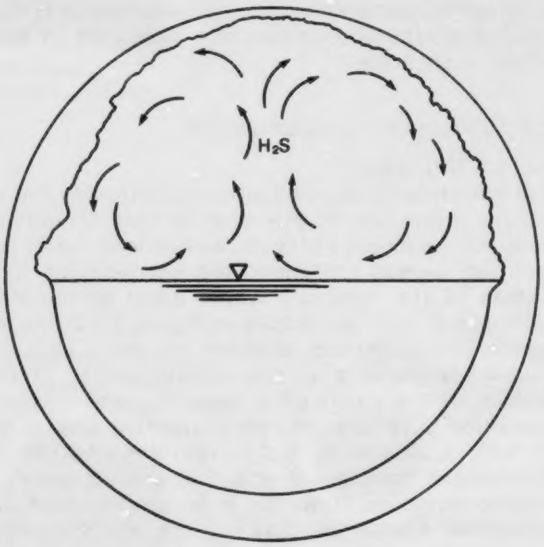
Bacteria play an important role, either directly or indirectly, in corrosion of materials in wastewater systems. For example, anaerobic sulfate-reducing bacteria can attack the protective sulfate coatings on metal and concrete and leave them vulnerable to corrosion by sulfuric acid resulting from oxidation of H_2S. H_2S oxidation to sulfuric acid in the presence of moisture also results from action of bacteria which colonize the moist slimes above the water surface. Other types of bacteria can destroy asphaltic coatings that are normally resistant to chemical attack, leaving the parent surface exposed and vulnerable to attack by chemical agents.

Another potential result of bacterial action is the formation of localized galvanic cells in an electrolyte such as wastewater. Bacterial colonization on surface slimes or in the liquid results in a depression of the pH in the immediate area of the bacteria. The pH change may result in a lower electrical potential, with the localized area acting as a cathode. The adjacent metal becomes an anode, and electrochemical corrosion may result (17).

2.4.3 Fatigue Corrosion

Virtually all ductile metals have a limit to the number of times they can be stressed or bent before cracking or breaking. When these metals are subjected to such stresses under corrosive conditions, this limit may be

Figure 2-13. **Distribution of corrosion in a sewer.**



reduced substantially. The process by which the stress limit is reduced is called fatigue corrosion. The actual corrosion mechanism that is responsible for this phenomenon may be oxidation, hydrogenation, direct chemical corrosion, or galvanic corrosion due to the heat and stresses generated within the metal. Oxidation, hydrogenation, and direct chemical corrosion may occur due to the separation of the grain boundaries, allowing penetration of the corrosive agents to the internal surfaces of the metal. Galvanic cells may be formed from electrical currents and differential pressures that result from frictional heat generation during slippage of the grain boundaries (17).

2.4.4 Stress Corrosion

Stress corrosion is similar to fatigue corrosion, except that in stress corrosion, the stress is generally preapplied and may result from temperature or strains induced by working. However, the mechanism (slipping or separation of grain boundaries) is much the same. The result is usually splitting or cracking of the metal.

Straining of metal also produces electrical currents which polarize the metal and increase its attraction to oxygen and other corrosion-inducing agents. Metals can retain this polarity and may thus be subject to much more corrosion than if straining had not occurred (17).

2.4.5 Fretting Corrosion

Fretting corrosion is a combination of wear (erosion) and the oxidation of the wear products on the freshly exposed metal. A good example of fretting corrosion in a wastewater treatment plant occurs due to the action of flights along the rails in the bottom of a clarifier. The wearing action of the metal shoes is accelerated by the presence of grit and abrasive material in the wastewater. The fresh metal surfaces exposed by this action are subjected to corrosion by the corrosive agents in the wastewater (17).

2.4.6 Cavitation Erosion

Cavitation erosion is normally associated with pump impellers, although it can occur at any point where high liquid velocities and sudden, violent reductions of fluid pressure exist. This can occur even in noncorrosive fluids. Several theories exist regarding this mechanism, including the following (17):

1. Sudden and extreme changes in pressure distort the metal surfaces, allowing penetration of oxygen or hydrogen into the lattice structure during periods of high pressure. Gas molecules combine and literally explode during periods of low pressure, breaking off minute sections of the metal surface.
2. Penetration of the lattice structure of the metal by the corrosive agent results in oxidation or hydrogenation of the interior surfaces, with subsequent erosion of the corrosion products due to the velocity of the liquid.
3. Galvanic cells form in the metal as a result of differential pressures in the liquid.

2.4.7 Filiform Corrosion

Filiform corrosion may occur on metal surfaces with organic coatings and is induced by pinpoint penetration of moisture at numerous points on the surface. Through chemical and electrochemical processes, the corrosion progresses in narrow lines beneath the coating. Oxygen and moisture penetrating the coating support corrosion and the subsequent growth of these filament-like grooves in the metal surface (17).

2.4.8 Electrochemical Corrosion

2.4.8.1 Bimetallic or Galvanic Corrosion

Galvanic corrosion occurs from the electrical current created when two or more dissimilar metals are immersed in an electrolyte. Although water or wastewater is usually the electrolyte of interest, moist soils or moist gases may also serve as electrolytes. The resulting current-generating cells are referred to as "dissimilar electrode cells," and may occur under a wide range of conditions. Examples of such conditions include use of brass or bronze valves with iron pipe, variation in chemical composition and moisture content of backfill materials surrounding a pipe, differences in dissolved gas concentrations in the electrolyte, and differences in temperature within a pipe.

The tendency of metals to enter into this type of reaction is due to a property referred to as electromotive force or electric potential. The following metals are listed in the order of decreasing electromotive force: magnesium, aluminum, zinc, chromium, iron, cadmium, nickel, tin, lead, hydrogen, copper, mercury, silver, platinum, and gold (43). When two metals form a dissimilar electrode cell, the metal with the highest electromotive force serves as the anode (negative polarity), while the other metal acts as the cathode (positive polarity). Factors which affect the rate of reaction include proximity of the two metals, conductivity of the electrolyte, temperature, and pH (17).

2.4.8.2 Parting

Parting occurs in alloys immersed in an electrolyte when dissimilar electrode cells are formed between the various metals in the alloy. When the corrosion products are eroded by the velocity of the liquid, certain metals may be removed from the alloy. This can substantially change the alloy's property, such as reduce its strength or ductility (17).

2.4.8.3 Electrolysis or Stray Current Corrosion

This type of corrosion is caused by stray or external currents of electricity passing through soil or water in which a metal object is submerged. Current enters the metal, travels along the metal as the path of least resistance, and leaves at what becomes the anode, where corrosion occurs. Stray current corrosion is not a significant concern at wastewater treatment plants, although it may be a problem with steel pipe used in wastewater collection systems (17).

2.5 Predicting Sulfide Buildup and Corrosion in Sewers

Prediction of the rate of sulfide buildup and corrosion potential is an essential element in the design of new sewer systems as well as in the evaluation of existing systems. Equations are presented in this section that can be used for this purpose. It should be noted, however, that several of the coefficients are empirically determined, and will vary significantly from one condition to the next. For existing systems, assumed coefficients can be used in the predictive models to estimate sulfide buildup or corrosion penetration.

2.5.1 Predicting Sulfide Buildup

2.5.1.1 Pipes Flowing Less Than Full

Several equations have been developed to predict the buildup of sulfides in both gravity sewers and force mains. The first, applicable only for lines flowing less than half full, determines the "marginal velocity" above which sulfide generation will not occur (12). Another equation incorporates wastewater flow depth and velocity (18). A further modification yields the "Z" equation, which can be used for gross estimates of sulfide generation potential (19).

The following equation was developed by Pomeroy and Parkhurst to account for other factors affecting sulfide buildup in a pipe flowing less than full (20). This equation applies only to pipes flowing less than full and in which little or no DO exists.

$$\frac{d[S]}{dt} = \frac{M' \, EBOD}{R} - \frac{m[S](su)^{3/8}}{d_m} \qquad (2\text{-}23)$$

where,

$\frac{d[S]}{dt}$ = rate of change of total sulfide, mg/l-hr

M' = effective sulfide flux coefficient for sulfide generation by the slime layer in gravity sewers, experimentally determined empirical constant, m/hr

$EBOD$ = effective BOD = $BOD_5 \times 1.07^{T-20}$, mg/l

T = wastewater temperature, °C

R = hydraulic radius, equal to area of flow divided by wetted perimeter (P), m

m = empirical coefficient to account for sulfide losses by oxidation and escape to atmosphere, dimensionless

$[S]$ = total sulfide concentration, mg/l

d_m = mean hydraulic depth, equal to area of flow divided by surface width (b), m

u = mean sewage velocity, m/s

s = slope of energy grade line, m/m

The first term on the right-hand side of the equation accounts for sulfide generation by the slime layer, and assumes that this is the sole source of sulfide. This assumption is fairly accurate, as sulfide generation within the stream of a gravity sewer is usually negligible. The second term accounts for losses of sulfide due to oxidation in the stream and emission to the sewer atmosphere.

At equilibrium, $d[S]/dt = 0$. Therefore:

$$\frac{M' \, EBOD}{R} = \frac{M[S](su)^{3/8}}{d_m}$$

Solving for [S], the theoretical upper limit at equilibrium, yields:

$$[S]_{lim} = (M'/m) \, EBOD \, (su)^{-3/8} (P/b) \qquad (2\text{-}24)$$

This limit will never be reached theoretically, but will be approached asymtotically.

The downstream sulfide concentration, S_2, at time t_2 can be predicted directly from the following:

$$S_2 = S_{lim} - \frac{(S_{lim} - S_1)}{\log^{-1}\left[\frac{m(su)^{3/8} t}{2.31 \, d_m}\right]} \qquad (2\text{-}25)$$

where,

S_2 = predicted sulfide concentration at time t_2, mg/l

S_1 = sulfide concentration at time t_1, mg/l

S_{lim} = limiting sulfide concentration from Equation 2-23, mg/l

s = slope, m/m

u = stream velocity, m/s

t = $(t_2 - t_1)$ = flow time in a given sewer reach with constant slope, diameter, and flow, hr

m = empirical coefficient for sulfide losses

d_m = mean hydraulic depth, equal to area of flow divided by surface width, m

This equation allows the input of initial sulfide concentration S_1, $(S_{lim} - S_1)$ being the initial sulfide deficit. Solving the equation for S_2 requires an estimate of flow time, t. This can be calculated from an estimate of the stream velocity, u. Velocities can be estimated using dyes, floating objects, velocity meters, or hydraulic computations. In one study, velocities were estimated using dyes, floating objects, and hydraulic computations. Most probable velocity values were then selected based on the tests conducted and the condition of the lines (21).

Two sets of assumed values for the coefficients M' and m have been suggested, with the following interpretation (20):

Moderately conservative: Low DO, sulfide buildup in progress

$M' = 0.32 \times 10^{-3}$
$m = 0.96$

More conservative: Observed sulfide buildup generally less than predicted

$M' = 0.32 \times 10^{-3}$
$m = 0.64$

These coefficients can be used as first estimates for Equations 2-24 and 2-25. Where possible, the equations can be calibrated based on data collected from existing systems. This will allow adjustment of coefficients to improve correlation between measured and predicted values. This is often done when it is desired to predict sulfide levels in proposed expansions to existing collection systems.

2.5.1.2 Pipes Flowing Full

In a force main or surcharged sewer trunk line, the pipe is flowing full, thus minimizing or eliminating surface reaeration, sulfide oxidation, and sulfide losses to the sewer atmosphere. The term on the far right of Equation 2-23 can be eliminated. The first term on the right-hand side of Equation 2-23 has been modified to account for sulfide generation in the stream, and assumes zero DO in the wastewater (20):

$$\frac{d[S]}{dt} = M \left[EBOD \left(\frac{4}{d} + 1.57\right)\right] \tag{2-26}$$

where,

$\frac{d[S]}{dt}$ = rate of change of total sulfide, mg/l-hr

M = sulfide flux coefficient for filled pipe, experimentally determined empirical constant, m/hr

EBOD = effective BOD = BOD x $1.07^{(T-20)}$, mg/l

d = pipe diameter, m

Integrating and solving for S_2 yields:

$$S_2 = S_1 + (M)(t) \left[EBOD \left(\frac{4}{d} + 1.57\right)\right] \tag{2-27}$$

where,

S_2 = predicted sulfide concentration at time t_2, mg/l

S_1 = sulfide concentration at time t_1, mg/l

t = $t_2 - t_1$ = flow time in a given sewer reach with constant slope, diameter, and flow, hr

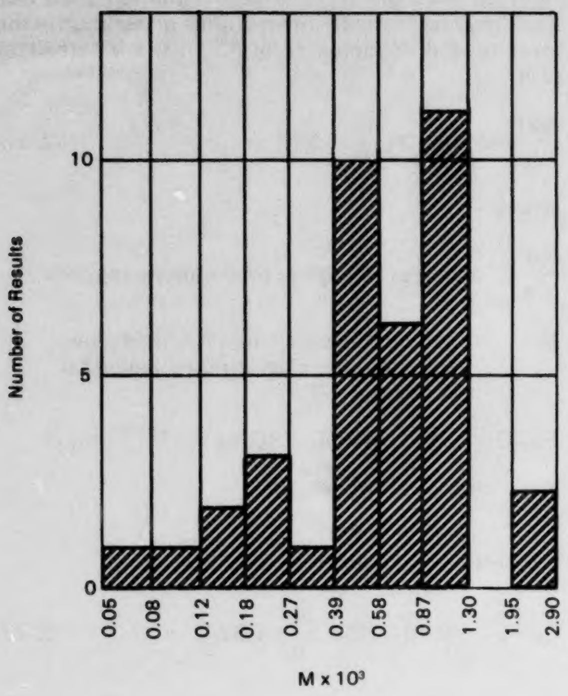
Figure 2-14 shows the distribution of empirically determined values for M. In general, a value of 1×10^{-3} m/hr is reasonable for force mains in which conditions are favorable for sulfide buildup (8).

Given the initial sulfide concentration, hydraulic radius of the pipe (d/4), EBOD, and an assumed value for M, it is possible to predict the sulfide buildup at any point downstream (at time t) in a pressure main.

2.5.1.3 Consideration of Junctions and Tributaries in Estimating Sulfide Buildup

Junctions and tributary sewers can cause changes in wastewater quality that affect the generation of sulfide in sewer systems. For example, a force main or pressure sewer discharging into a gravity line may significantly increase the sulfide concentration in the

Figure 2-14. Specific sulfide flux coefficients from filled-pipe data (8).



gravity sewer. In addition, turbulent junctions may strip H_2S to the sewer atmosphere and add DO to the wastewater. In general, where sulfide is likely to be present, junctions should be designed to minimize turbulence, and thus releasing of H_2S, in order to avoid odor and corrosion problem at that point.

In estimating sulfide buildup in a sewer trunk, the affect of tributary flows can be handled in the following manner:

1. Calculate the estimated sulfide buildup for each of the tributary sewers using Equation 2-25 (gravity mains) or Equation 2-27 (force mains).
2. Calculate the starting sulfide concentration, S_1, in the trunk line at each junction using simple mass balance relationships.

For example, at a given junction:

Q_{trunk} = 4.0 cfs (before junction)

$[DS]_{trunk}$ = 2.0 mg/l (before junction)

Q_{trib} = 1.0 cfs

$[DS]_{trib}$ = 1.0 mg/l

$$S_1 = \frac{(Q_{trunk})([DS]_{trunk}) + (Q_{trib})([DS]_{trib})}{Q_{trunk} + Q_{trib}}$$

$$S_1 = \frac{(4.0)(2.0) + (1.0)(1.0)}{4.0 + 1.0}$$

$$S_1 = 1.8 \text{ mg/l}$$

3. After calculating a new starting sulfide concentration, S_1, at each junction, calculate the sulfide concentration for each downstream reach.

2.5.1.4 Example

The following example illustrates the methodology for predicting downstream sulfide concentrations in a gravity sewer.

DATA: Single gravity sewer trunk with no other contributing flows.

d = diameter = 0.91 m

s = slope = 0.001

depth of flow = 0.45 m

u = velocity = 0.61 m/sec

M' = 0.32×10^{-3}, assumed

m = 0.96, assumed

S_1 = 0.5 mg/l

BOD_5 = 200 mg/l

T = 25°C

Calculate sulfide concentration for a t of 5 hrs.

$$[S]_{lim} = (M'/m)\ EBOD\ (su)^{-3/8}\ (P/b) \qquad (2\text{-}24)$$

$$[S]_{lim} = \left[\frac{.32 \times 10^{-3}}{0.96}\right] [200\,(1.07)^{25-20}] \; [(0.001)(0.61)]^{-3/8} \left[\frac{1.43}{0.91}\right]$$

$$[S]_{lim} = 2.36 \text{ mg/l}$$

$$S_2 = S_{lim} - \frac{(S_{lim} - S_1)}{\log^{-1}\left[\dfrac{m(su)^{3/8}\,t}{2.31\ d_m}\right]} \qquad (2\text{-}25)$$

$$S_2 = 2.36 - \frac{(2.36-0.5)}{\log^{-1}\left[\dfrac{(0.96)\,[(0.001)(0.61)]^{3/8}(5)}{(2.31)(0.36)}\right]}$$

$$S_2 = 2.36 - \frac{1.86}{\log^{-1}(0.360)}$$

$$S_2 = 2.36 - 0.81$$

$$S_2 = 1.55 \text{ mg/l}$$

2.5.2 Predicting Rates of Corrosion

2.5.2.1 Corrosion Rate Predictive Model

The rate of corrosion of cementitious pipes can be presented theoretically by equations. The corrosion rate depends upon the rate of sulfuric acid production and the alkalinity of the pipe material. Sulfuric acid production is related to the mass emission of sulfide from the wastewater. The corrosion rate of cement bonded pipe can be estimated by assuming the rate at which H_2S will reach the pipe wall and the amount reaching the wall that will be oxidized and available for reaction. Thirty-two grams of sulfide is required to produce the sulfuric acid to dissolve 100 grams of alkalinity expressed as $CaCO_3$. The corrosion rate equation developed by Pomeroy is (8):

$$C_{AVG} = \frac{11.5\, k\, \phi_{sw}}{A} \qquad (2\text{-}28)$$

where,

C_{AVG} = average rate of penetration, mm/yr

k = Coefficient of efficiency for acid reaction considering the estimated fraction of acid remaining on the wall. May be as low as 0.3 and will approach 1.0 for a complete acid reaction.

ϕ_{sw} = flux of H_2S to the pipe wall, gm^2-hr

A = Alkalinity of the cement bonded material, expressed as $CaCO_3$ equivalents. Approximately 0.18 to 0.23 for granitic aggregate concrete, 0.9 for calcareous aggregate, 0.4 for mortar linings, and 0.5 for asbestos cement.

11.5 = constant

$$\phi_{sw} = 0.69\,(su)^{3/8}\, j\,[DS]\,(b/P') \qquad (2\text{-}21)$$

where,

s = energy gradient of wastewater stream, m/m

u = stream velocity, m/s

j = fraction of dissolved sulfide present as H_2S as a function of pH (see Figure 2-3)

[DS] = average annual concentration of dissolved sulfide in the wastewater, mg/l

b/P' = ratio of width of wastewater stream at surface to exposed perimeter of the pipe wall above the water surface

Rates of corrosion are usually expressed in millimeters of penetration per year measured inwards from the original interior profile. Areas where the rate of corrosion reaches a maximum may show greater rates of penetration than the average condition. It has been suggested that the most rapid attack may be 1.5 times the average (8). Note that the dissolved sulfide concentration used in Equation 2-28 is the annual average sulfide concentration, not the peak or climactic concentration. Average annual dissolved sulfides may be only 25 to 50 percent of the climactic values. A thorough discussion of average annual vs. peak dissolved sulfide concentrations may be found in reference 22.

The choice of the coefficient of efficiency for the acid reaction, k, is a matter of engineering judgment. K will approach unity when the rate of acid production is very slow, and may be as low as 0.3 to 0.4 if acid production is rapid and if much condensate is formed, as with warm wastewater flowing in a cold pipe. In large pipes with moderate rates of acid formation, most of the acid will react, and k will most likely be in the range of 0.9 to 1.0.

2.5.2.2 Example

DATA: Reinforced concrete pipe with granitic aggregate; 25-mm cover over reinforcing steel

d = diameter = 1.07 m

s = slope = 0.00088

y_o = depth of flow = 0.214 m

pH = 7.0

k = coefficient of efficiency for acid reaction, assumed to be 0.8

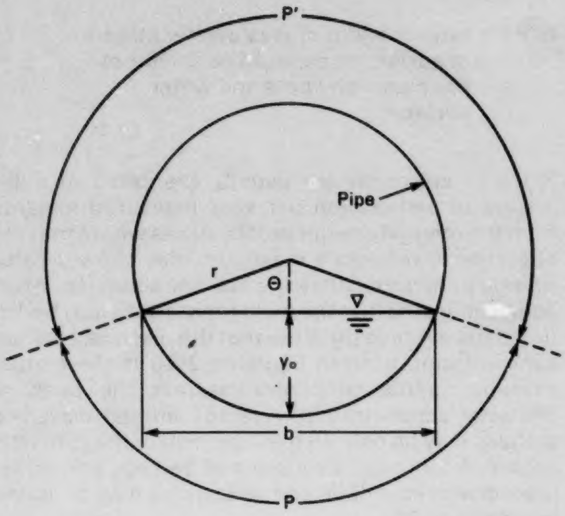
A = measured alkalinity of concrete = 0.2 (granitic aggregate)

[DS] = average annual dissolved sulfide concentration = 2.0 mg/l

CALCULATE:

1. Corrosion rate
2. Expected lifetime of pipe

First, calculate hydraulic factors:



$$R = \frac{\text{Area}}{P};$$

$$\text{Area} = 3.14r^2\,(\theta/180) - (b/2)\,(r-y_o)$$

where,

$$\theta = \cos^{-1}\frac{(r-y_o)}{r}$$

$$\theta = \cos^{-1}\left[\frac{(0.535 - 0.214)}{0.535}\right] = 53.1^\circ$$

$$b = 2[r^2-(r-y_o)^2]^{0.5}$$

$$b = 2[(0.535)^2 - (0.535-0.214)^2]^{0.5} = 0.86 \text{ m}$$

$$\text{Area} = [3.14\,(0.535)^2\,(53.1/180)] - [(0.86/2)\,(0.535-0.214)] = 0.127 \text{ m}^2$$

$$P = 2\,(3.14)\,(r)\,(\theta/180)$$

$$P = 2\,(3.14)\,(0.535)\,(53.1/180) = 0.99 \text{ m}$$

$$P' = 2\,(3.14)r - P$$

$$P' = (2)\,(3.14)\,(0.535) - 0.99 = 2.37 \text{ m}$$

$$R = \frac{\text{Area}}{P} = 0.127/0.99 = 0.128 \text{ m}$$

$$V = \frac{R^{2/3}S^{1/2}}{n}$$

(Manning equation; assume n = 0.013)

$$V = \frac{(0.128)^{2/3}\,(0.00088)^{1/2}}{0.013} = 0.58 \text{ m/s}$$

(Alternatively, V can be based on actual measured values.)

Next, calculate ϕ_{sw} and C_{avg}:

$$\phi_{sw} = 0.69(su)^{3/8}\,j[DS](b/P')$$

From Figure 2-3, at pH = 7.0, j = 0.5

$$\phi_{sw} = 0.69\,(0.00088 \times 0.58)^{3/8}\,(0.5)\,(2.0)\,(0.86/2.37)$$

$$\phi_{sw} = 0.015 \text{ g/m}^2\text{-hr}$$

$$C_{avg} = \frac{11.5\,K\,\phi_{sw}}{A} \quad \text{(Assume K = 0.8)}$$

$$C_{avg} = \frac{11.5\,(0.8)\,(0.015)}{0.2} = 0.69 \text{ mm/yr}$$

Expected life of pipe (based on exposure of reinforcing steel)

= 25 mm/0.69 mm/yr = 36.2 years

The preceding calculation is based on the assumption that the life of the pipe is equal to the time required for acid corrosion to reach the reinforcing steel. If the expected life is less than the desired design life, changes to such parameters as pipe slope, alkalinity, cover thickness, etc. should be considered.

2.5.3 Case Study for Predicting Sulfide Buildup and Corrosion

In 1976, the county of Sacramento, California, conducted an extensive sulfide investigation of 13 miles of gravity sewer trunk (Central Trunk). The sewer was placed into operation in 1963, and total and dissolved sulfides have been continuously monitored since 1965. Visual inspections were conducted in 1964, 1968, 1969, and 1976. Severe corrosion was not apparent until 1974, which spurred a more extensive sampling program to establish the cause and extent of sulfide buildup and corrosion.

Table 2-7 provides a comparison of predicted and measured total sulfide concentrations in the Sacramento Central Trunk (9). The predicted values were calculated to account for gains and losses of sulfide from tributary streams. Note that the correlation is good when the coefficient m in Equation 2-25 is assumed to be 0.96.

A comparison was made between the actual measured corrosion penetration and that predicted by the corrosion rate equation (Equation 2-28). Alkalinity was measured to be 0.16 and k was assumed to be 1.0. The sulfide flux rate, ϕ_{sw}, was calculated based on 15 years of operating data, which included dissolved sulfide concentrations, flows, pH values,

Table 2-7. **Comparison of Predicted and Measured Total Sulfides in Sacramento Central Trunk (9)**

Distance Upstream from CWTP[a]	Total Sulfides	
	Predicted[b]	Measured[c]
ft	mg/l	
8,000	1.5	1.4
22,000	1.8	1.5
35,000	1.9	1.8
55,000	1.4	0.9

[a] Sampling stations in the Central Trunk located upstream from the Central Wastewater Treatment Plant (CWTP).

[b] Predicted using Equation 2-25 with m = 0.96 and considering the effect of junctions.

[c] Represents the average of 48 samples.

and sewer gradients at manholes. Expandable rods were used to measure pipe diameters. Soft concrete was chipped away to hard concrete to determine the depth of acid penetration. The measured corrosion penetration for the various reaches of the trunk sewer are shown in Table 2-8 (9).

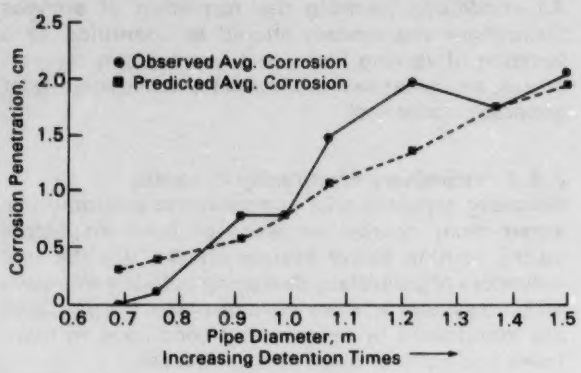
Table 2-8. **Comparison of Measured vs. Predicted Corrosion Penetration, Sacramento, CA, Central Trunk Sewer (9)**

Pipe Diameter	Pipe Length	Avg. Corrosion Penetration	
		Measured	Predicted*
m	m	cm	cm
0.69	823	0	0.3
0.76	1,458	0.1	0.4
0.91	2,792	0.8	0.6
0.99	1,778	0.8	0.8
1.07	14,838	1.5	1.1
1.22	3,703	2.0	1.4
1.37	2,765	1.8	1.8
1.52	3,738	2.1	2.0

*Using Equation 2-29 with A = 0.16
k = 1.0

The predictive equation (Equation 2-28) compared well with the actual measured corrosion in the trunk sewer. The results are shown graphically in Figure 2-15.

Figure 2-15. **Observed and predicted corrosion penetration, Sacramento, CA central trunk sewer.**



The utility in comparing existing sulfide levels and corrosion rates to predictive models for sulfide generation and corrosion is best demonstrated when an existing system in the same city or area is undergoing expansion. For design purposes, an analysis can be conducted of the existing system to obtain sulfide levels and corrosion rates. Data from a system in close proximity to, or part of, the proposed system are preferred since climate, terrain, and wastewater characteristics are likely to be similar. The measured corrosion rates can be used to calibrate the predictive models for subsequent use in the design and evaluation of new or expanded systems, as well as for the design of corrective measures for the existing systems. The models yield only estimates and should be used accordingly. The accuracy of the model is dependent upon the choice of empirical coefficients which, when possible, should be based on historical data. The use of predictive models in design of new collection systems is described in detail in Chapter 5.

2.6 Approach to Investigating Odor and Corrosion

A survey of a wastewater collection system and treatment works is necessary to identify the sources and causes of odor generation. A number of monitoring stations should be selected within the collection system based on area served; length, diameter, flow and hydraulic gradient of the sewers; location of force main discharges; and other site-specific factors. Treatment plants should be monitored at the plant head works, at points of sidestream returns, and at other locations where sulfide generation or release is likely to occur. Collection systems should be monitored at lift stations, force main discharge points, junctions of major collectors and interceptor sewers and at points where the sewer system undergoes major changes in average slope.

Wastewater quality and water levels should be monitored at each selected station. Samples should be taken and analyzed both in the field and in the laboratory for selected parameters. Measurements of atmospheric H_2S at force main discharge points, at transition manholes and sewage lift stations, and in enclosed areas exposed to wastewater and sludge should be included as critical elements of the survey. All conditions favoring the formation of sulfides throughout the system should be identified as a function of varying flow rates, wastewater temperatures, and other seasonal conditions that affect odor generation potential.

2.6.1 Preliminary Monitoring Program

Normally, repeated odor complaints in a community, either from nearby residents or from inspectors during routine sewer system checks, are the first indicators of potentially damaging sulfide generation within a system. In more extreme cases, the problems are manifested by deteriorated conditions in manholes and pipes or by structural failures.

Early evidence of sulfide generation in existing systems warrants the implementation of at least a preliminary program to assess the overall potential for sulfide generation. Such a preliminary program should include a thorough investigation of odor complaints, and a systematic investigation of the collection and treatment system to identify major potential contributors such as force main discharge points, critical sewer reaches and juncture points, and possible industrial sources. The preliminary investigation will require knowledge of past complaints as well as an up-to-date record of sewer maps and flow information.

2.6.1.1 Collection System and Treatment Plant Schematics

A collection system map should be available, including sizes and types of pipes, slopes of lines, flows and accurate manhole locations. This information can be reduced to a one-line schematic diagram for easy reference and for recording subsequent data. The diagram should provide information on major pumping stations; trunk, tributary and collector sewers; pipe diameters; pipe lengths; flow rates; and gravity and force main locations. The frequency of pump operation and wet-well sizes should also be determined at this stage of an investigation. Similarly, schematic diagrams and site maps of the treatment plant may allow preliminary identification of potential sources of odor and corrosion.

2.6.1.2 Preliminary Sampling

The survey of odor generation should begin at the wastewater treatment works and proceed upstream throughout the system. The preliminary survey should consist of a simplified field analysis of sulfide levels to determine the actual trouble areas. A field survey crew can use a portable kit in which a sample is collected, an effervescent agent is added to liberate the H_2S, and lead acetate paper discs are used to detect the presence of sulfides in the wastewater (23). A comparison chart is used to determine the approximate concentrations up to 5 mg/l. With a 1 to 1 dilution, the analysis is acceptable up to 10 mg/l of H_2S. This technique takes about 5 minutes per sample so that the preliminary survey can progress rapidly. Normally, odor problems will occur when dissolved sulfide levels are 1.0 to 1.5 mg/l or greater, although some communities have reported odor problems with dissolved sulfide levels of only 0.1 mg/l.

Working upstream in the collection system using the preceding gross quantitative sulfide measuring technique is a rapid and effective method to isolate problem areas requiring further investigation. The following locations should be checked:

1. Pump stations—sample wet-well influents, pump discharges, and ends of pressure mains to determine sulfide balance.
2. Force main discharges into gravity sewers—turbulence may release H_2S gas and cause corrosion of manhole chambers.
3. Junctions and tributaries—sample tributaries to determine contribution of sulfides. Tributary flows with little or no sulfides and some DO may be beneficial in reducing sulfide problems.
4. Areas of turbulence and long detention times—sample locations of turbulence where sulfide may be released. Measurement of dissolved H_2S before and after the point of turbulence can give an indication of the quantity of H_2S released to the atmosphere. Also, check sewer reaches and wet wells with long detention times where sulfide generation is favored.

At a wastewater treatment plant, a preliminary survey would entail collection of samples from upstream of the headworks, downstream of preliminary treatment facilities, and downstream of primary clarifiers. Recycle streams such as supernatants from thickeners, digesters, and other sludge treatment processes should also be sampled, as these can contain high concentrations of H_2S that can result in severe odor and/or corrosion problems at the point of sidestream return. Air samples from enclosed spaces exposed to wastewater may also be collected to determine the severity of odors.

2.6.2 Detailed Evaluations

2.6.2.1 Sampling

The preliminary program will identify the locations for further sampling points and, in some cases, such as in very small communities, limited additional sampling may be all that is needed to quantify sulfide levels. In larger systems, more information would likely be required. Sulfide levels vary with diurnal flow rate so it is important to sample at different times of the day. Design of odor control systems is based on maximum and minimum conditions, not just average sulfide levels. Ideally, 24 discrete hourly samples should be collected over a period of 2 to 5 days, and the results plotted to reflect sulfide levels with a time cycle, flow cycle, and day-to-day cycle. Samples should be collected so that no aeration of the sample occurs. Field analyses should be performed immediately upon sample collection. Samples that are to be transported to the laboratory for sulfide analyses should be preserved immediately upon sample collection by adding a zinc acetate solution and by sealing the sampling container when completely full of liquid.

A mass sulfide profile should be prepared for each interceptor sewer entering the treatment plant as shown in the example in Figure 2-16 for the Orange County Sanitation District Plant No. 2, Orange County, California (24). The mass profile is used to design proper systems to control odor and corrosion.

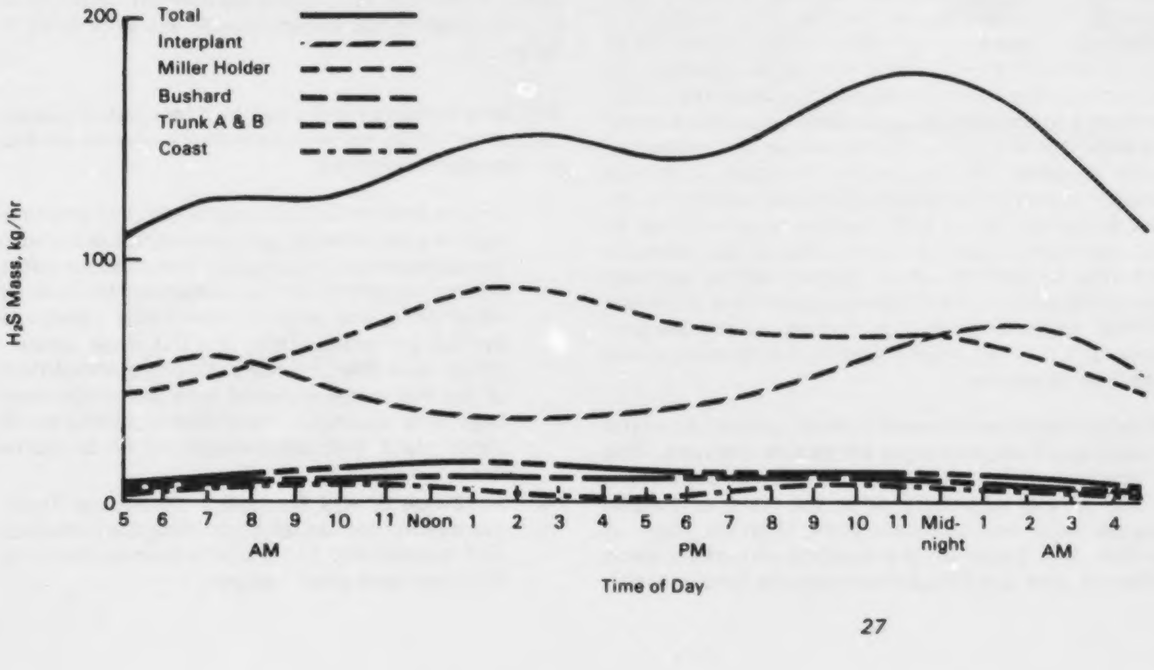
2.6.2.2 Analyses

The wastewater should be analyzed for the following:

1. Sulfates ($SO_4^=$), mg/l
2. pH
3. Dissolved Sulfides [DS], mg/l
4. Total Sulfides [TS], mg/l
5. Biochemical Oxygen Demand (BOD), mg/l
6. Dissolved Oxygen (DO), mg/l
7. Temperature, °C
8. Oxygen Depletion Rate (ODR), mg/l-hr.
9. Suspended Solids (TSS), mg/l

These parameters are all needed for properly assessing potential odor and corrosion problems and for utilizing the predictive models presented in Section 2.5 of this manual. In addition to these parameters, the pH of the crusty moist area on pipe walls or structures should be taken to determine if a corrosive environment exists. This can be accomplished by pressing litmus paper against the interior wall of the pipe (21). It is also useful to measure the ORP of the wastewater. A 1981 H_2S control study for the Kailua sewage collection system on the island of Oahu, Hawaii used DO and ORP as criteria to evaluate the effectiveness of preventing sulfide generation, and dissolved sulfides were used to evaluate the effectiveness of the odor abatement measures to remove sulfide from the wastewater (25). ORP is a measure of relative concentrations of oxidants (oxygen, nitrate, sulfate, etc.) and reductants (ammonia, sulfides, organics, etc.) in a system. An anaerobic biological system displays an ORP lower than that of an aerobic system. A value of +100 millivolts was set as the minimum ORP required at the various Kailua sewage pumping stations, transition manholes and end-of-line manholes to effectively prevent sulfide generation throughout the downstream portions of the Kailua collection system. The critical value for ORP is dependent on the character of the wastewater and the configuration of the collection system, and it may be different for each system.

Figure 2-16. **Composite H_2S mass profile entering Plant No. 2, Orange Co., CA (24).**



2.6.2.3 Analytical Techniques

Samples of wastewater should be analyzed for total and dissolved sulfides. Total sulfide analysis measures the H_2S and HS^- species present, and any acid-soluble metal sulfides contained in the suspended solids. Dissolved sulfide analyses determine the sulfide remaining in the wastewater upon removal of the suspended solids by filtration. The samples may be analyzed in the field using commercially available test kits, or may be transported to the laboratory for analyses.

Qualitative field analysis is useful for determining the presence of H_2S in wastewater samples. Several methods of field analysis are applicable for sulfide detection. The antimony test, silver sulfide-silver electrode test, lead acetate paper, and silver foils are all described in Standard Methods for the Examination of Water and Wastewater (14th Edition). Field tests are also available that will give quantitative as well as qualitative sulfide results. These kits use a colorimetric sulfide determination and can be used for either dissolved or total sulfide, depending on sample preparation.

Laboratory analysis of wastewater samples for sulfide can be accomplished using either the methylene blue colorimetric method (0 to 20 mg/l range) or the iodine titration technique (1 to 20 mg/l range). There are substances that react with iodine, such as thiosulfate, sulfite and various organic compounds, that will interfere unless removed. The iodine titrimetric method is used to standardize the colorimetric method.

A novel method of sulfide analysis that uses an indicator tube and a sealed sample container has recently been developed (26). In this method, the sample is added to a buffering solution with a pH of 5.0 in a flask fitted with a two-holed stopper. An H_2S-indicating tube is inserted into one hole of the stopper and a glass tube extending below the sample surface is inserted into the other hole of the stopper. After agitating the sample for 1 minute, a known volume of sample is pulled through the indicator tube. The concentration of total sulfides is determined by comparing the scale reading on the indicator tube to a pre-established calibration curve made by applying the same procedures to standard solutions of known sulfide concentration. This technique can be performed in the field once a standard calibration curve has been prepared.

Most professional surveys incorporate the use of the methylene blue technique for sulfide analysis. This procedure, which takes about 30 minutes per sample either in the laboratory or in the field, is simple, inexpensive, and accurate from 0 to 20 mg/l of sulfide. It is based on the reaction of sulfide, ferric chloride and p-aminodimethylaniline under conditions that produce methylene blue. A commercially available portable test kit is commonly used that is a variation of the methylene blue technique and which employs a matched set of test tubes. The blue color is developed in one test tube using the test water, which is then matched to a control solution in the other. The color in the second tube is developed with drop-by-drop addition of two standardized methylene blue solutions. This method substitutes N-diethyl-p-phenylene diamine oxalate (DPD) for N, N-dimethyl-p-phenylenediamine, which yields a more sensitive result (27).

The monitoring of atmospheric conditions in manholes, pumping stations, enclosed preliminary treatment works, and sludge handling buildings should include the identification of H_2S gas. The gas concentration can be determined directly by several portable methods:

1. Photoionization
2. Colorimetric detection tubes
3. Metal oxide semiconductor
4. Electrochemical sensor

These methods are described in Section 2.8.3.

2.6.3 Interpretation of Results

Serious odor or corrosion problems can be identified and isolated from the survey of collection systems and treatment works. This survey primarily identifies sources of sulfide generation; areas of release of H_2S which cause odor complaints; toxicity problems and direct corrosion of metals; and, through oxidation to sulfuric acid, corrosion of sewage collection system pipes, treatment works and equipment. Guidelines for interpreting the survey results are presented in Table 2-9.

The data compiled are used in odor control design decisions. Three general catagories of odor control are usually considered:

1. *Source Control.* Source control requires preventing the entry of materials not commonly found in typical domestic wastewater that produce odors during transport in the collection system, or eliminating one or more conditions necessary for H_2S generation (e.g., low DO, high temperature, favorable pH). For example, management of the discharge of septic tank pumpings (septage) into upstream manholes or at the treatment plant may be considered as a source control measure.
2. *Inhibition of H_2S Formation (In-Stream Treatment).* Alternatives for controlling the formation and release of H_2S in the collection system or at the treatment plant include:

a. Improving the oxygen balance
   - compressed air injection
   - pure oxygen injection

b. Chemical treatment, including the addition of:
   - chlorine
   - hydrogen peroxide
   - nitrates
   - metallic ions
   - lime

Table 2-9. Interpretation of Results of Wastewater Survey

Constituent	Interpretation
Sulfate	Sulfate will be reduced to sulfide under anaerobic conditions. Potential for sulfide generation rarely affected by sulfate concentration.
pH	At lower pH values, a greater proportion of molecular hydrogen sulfide is present that can be released to the atmosphere. A pH of 1 to 6 is conducive to sulfide generation and H_2S release.
BOD_5 and DO	Wastewater with a high oxygen demand rapidly takes up available DO and can create anaerobic conditions favoring sulfide generation. DO of at least 1.0 mg/l is desirable to prevent sulfide generation.
Dissolved Sulfide and Total Sulfide	Dissolved sulfides of 1.0 to 1.5 mg/l (lower in some cases) will normally contribute to odor and corrosion problems. Dissolved sulfides are usually 70 to 90 percent of total sulfides present. Sulfide levels are used as a criterion to determine effectiveness of odor control techniques.
Temperature	Higher temperature favors the biochemical generation of sulfides and lowers their solubility.
Atmospheric H_2S	H_2S has a very low odor threshold and is a toxic gas. Liberation of H_2S may be extremely variable depending upon conditions. H_2S gas emissions lead to corrosion problems. Concentration of atmospheric H_2S determines design parameters for air scrubbers and oxidizers for odor control.
Oxidation Reduction Potential (ORP)	Only at ORP's below zero does reduction of sulfates take place. Anaerobic systems display ORP's lower than aerobic systems.

ORP millivolts	Condition
+50	No action by anaerobic bacteria
0	Poor anaerobic activity
-100 to -200	Maximum efficiency for anaerobic activity
-50 to -300	Favored by sulfate-reducing bacteria for production of sulfides

3. *Collection and Treatment of Foul Air.* Exhaust air odor control systems are commonly designed to create negative air pressures in an enclosed area and to treat the air before exhausting to the atmosphere. Atmospheric H_2S should be reduced to less than 1 ppm before discharge. Specific treatment methods are discussed in subsequent chapters of this manual. The cost effectiveness of this approach vs. in-stream treatment is dependent upon pumping station locations, collection system layout, pipe sizes and wastewater flow rates, and the concentrations of H_2S in the wastewater and in the atmosphere of enclosed spaces.

2.7 Measurement and Monitoring of Corrosion and Odor

2.7.1 Corrosion

Corrosion is an ever-present problem in many wastewater collection and treatment systems, and prevention of corrosion can result in significant cost savings. Acid attack of collector or interceptor sewers is a significant cause of early pipe failure (28). The corrosion process discussed in this chapter is that which affects the surfaces of concrete, mortar, some metals, and other structural materials when they are exposed to a humid atmosphere containing H_2S.

2.7.1.1 Corrosion of Concrete

Characteristic features of corroding concrete surfaces are high sulfuric acid concentrations with low pH values. Acid attack is confined to the interior, unsubmerged portion of the sewer pipe and is heaviest at the crown and just above the liquid level. Concentrations of sulfuric acid can reach 5 percent, and a high percentage of this acid will react with the exposed surfaces and be neutralized by the alkalinity of the concrete. An acid attack situation can easily be demonstrated by testing the wall crust for pH.

An existing system can be examined for corrosion with proper planning and analysis. A sewer system map should be prepared for detailed assessment. The information should include age, size, and types of pipes; slopes of the lines; wastewater flows; and accurate manhole locations. Manholes should be examined for corrosion effects. An expandable probing rod can be used to measure internal diameters for comparison with originally installed diameters. Soft deteriorated concrete should be first chipped away to determine the depth of corrosion penetration. Core samples should be taken to determine wall thickness, alkalinity, and compressive strength at various locations along the sewer. The core samples need to be taken above and below the normal water line for comparison of thickness, because below the water line corrosion would not be expected. If proper core samples cannot be removed, the inside of the sewer pipe can be measured for thickness with a probing rod placed inside the drilled hole in the pipe.

2.7.1.2 Corrosion of Metals

Metallic components of wastewater treatment plants should be periodically inspected to detect the presence and severity of corrosion, since corrosion can adversely affect the properties of the metal. For example, the structural integrity of iron and steel deteriorates in the moist, oxidizing atmospheres of wastewater treatment plants (29). Copper contacts and components of electrical systems may be rapidly oxidized to black copper sulfate in the presence of only small amounts of H_2S (4 to 8 ppm), resulting in a weakened, poorly conducting material.

Structural components such as bridge work, bolts in concrete, gratings, wet-well steps and ladder rungs, and walkways should be carefully inspected for the effects of corrosion. The severity of the corrosion should be noted. Other metallic components such as bar screens, conveyor mechanisms, railings, and steel tanks should also be inspected as part of the investigation. Enclosed areas exposed to wastewater such as wet wells, preliminary treatment works, and sludge handling buildings generally harbor conditions favorable to corrosion due to the presence of moisture and corrosive gases such as H_2S. Corrosion of metal surfaces has been noted to be extremely severe for some plants in which preliminary treatment facilities have been enclosed without proper ventilation and dehumidification.

2.7.2 Odors

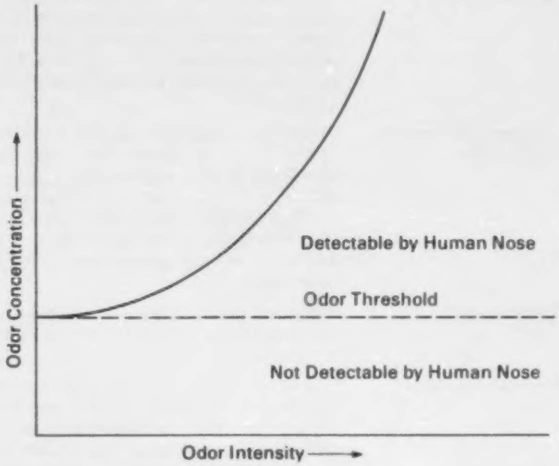
Measurement and characterization of odors is not only important in assessing the magnitude of the problem; it is also crucial for the proper design of odor control systems. For systems which remove odorous compounds from the air, such as wet scrubbers and activated carbon processes, accurate data on atmospheric concentrations of odorants are essential. Numerous cases exist in which failure to accurately measure peak concentrations of odorants led to underdesign of the treatment unit and poor performance.

2.7.2.1 Odor Threshold

An odor concentration level below which malodorous substances are no longer detectable by the human nose is defined as the threshold odor number (TON). The sensitivity to odors varies from individual to individual and, as such, odor thresholds are more subjective than objective in nature. When the inorganic gases or organic vapors, such as those described in Section 2.2, contact the human olfactory fibers, the sensation known as odor is created (3). All odor threshold values are determined using several persons on an odor panel to decide when the odor is no longer detectable. Low odor thresholds, low molecular weights and high vapor pressures suggest that a compound, if present in a malodorous atmosphere, can be expected to contribute to the objectionable odor. Odors can be contributed from a mixture of compounds.

After the odor concentration reaches and then exceeds the odor threshold, the odor intensity increases rapidly at first, then changes only slowly with further odor concentrations (Figure 2-17). Finally, the nose becomes insensitive and the person may no longer be conscious of the odor. This is especially critical in the case of H_2S, since loss of sensitivity to the odor can mean that the gas has reached a dangerous level.

Figure 2-17. **Typical concentration—intensity relationship for odors.**



2.7.2.2 Odor Measurement

To control odors, it is necessary to identify the odorous components of the vapors and gases causing a nuisance. However, positive identification is a difficult task and many times requires the use of a gas chromatograph (GC). With the GC, the procedure is to separate the gaseous components of air, make educated judgments as to the nature of the compounds, and then inject standards in order to match the compound in question with the pure compound. Because an odor is usually defined in physiological or psychological responses, the human nose is still used to identify and measure the intensity of odor. Although H_2S can be monitored effectively with instrumentation, other malodorous substances cannot be.

Therefore, the odor panel is the most common method used to identify odor nuisances.

The odor panel normally is made up of a given number of persons, usually eight or more. A sample is collected in a glass sampling bulb (25 to 1000 ml size) and delivered immediately to the odor panel for a series of dilutions and sniffings. The most widely accepted technique for odor measurement is the triangle olfactometer method, in which three samples are presented to the panelist from a series of glass sniffing ports. Two are test room air (blanks), and the third is odorous air diluted with test room air. The olfactometer supplies six dilution levels. The volumetric amount of the odorous gas which is detectable by only half the odor panel (of eight or more persons) in 0.03 m^3 (1 cu ft) of odor-free air is called an odor unit. The strength of an odor is determined by the number of dilutions with odor-free air needed to reduce an odor to a barely detectable level (30). The odor unit can be used as a reference point to compare to other systems, or to compare on a routine basis with a particular odor nuisance. Some state regulations are based on the odor unit as well. If odor control measures are instituted, the odor unit can be used to determine effectiveness of the odor control system by comparison with pre-control odor units. Caution must be observed in using an odor panel, and local air pollution control agencies should be consulted as to their required practices.

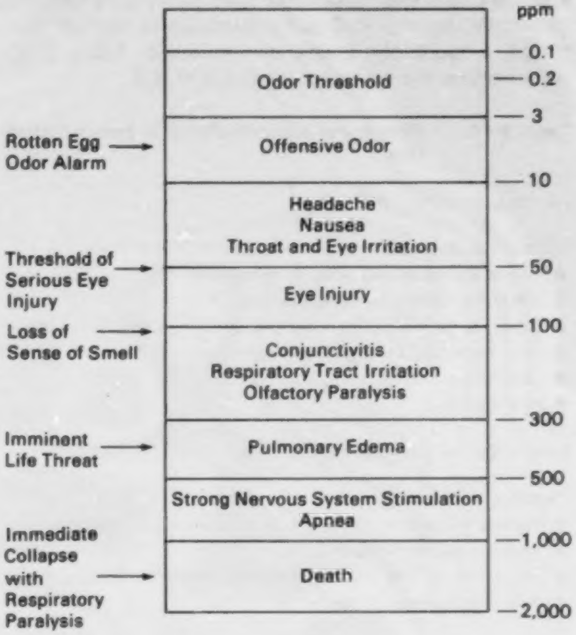
2.8 Toxicity and Safety Practices

H_2S is an acutely toxic material and has been responsible for the death of a number of sewer system workers. H_2S is heavier than air and therefore can be found in the lower portion of manholes. This deadly gas, whose toxicity has been ranked with hydrogen cyanide, is colorless and has a characteristic rotten egg smell at low concentrations. But as the levels of H_2S increase, workers are generally unaware of its presence. A person's ability to sense dangerous concentrations by smell is quickly lost. If the concentration is high enough, unconsciousness will come suddenly, followed by death if there is not a prompt rescue. Many times, rescue attempts are ill-fated since a person may not consider his own safety in trying to save a co-worker's life.

2.8.1 Dangers of Hydrogen Sulfide

The physiological effects of H_2S are summarized in Figure 2-18 (11). Very low concentrations of this gas can cause serious health hazards. Death has resulted from concentrations of 300 ppm by volume in air (28). Such concentrations can be obtained in an enclosed chamber with high turbulence, from wastewater containing 2 mg/l of dissolved sulfide at a pH of 7.0. Based on Henry's Law, Figure 2-9 was developed to show H_2S levels in the atmosphere in equilibrium with the given concentrations of H_2S in the water at the respective wastewater temperatures.

Figure 2-18. Hydrogen sulfide toxicity spectrum (11).



Concentrations of toxic gases to which a worker may be exposed can be expressed in several ways:

1. *Eight-Hour Time Weighted Average (TWA).* The maximum average concentration to which a worker can be exposed for 8 hours a day, 40 hours a week. This is normally called the threshold limit value (TLV).
2. *Ceiling Value.* A limit generally not to be exceeded.
3. *Acceptable Maximum Peak.* A concentration limit which is not acceptable for specified maximum duration.

The Occupational Safety and Health Administration (OSHA) has established limits for exposure to H_2S of 20 ppm (15-minute exposure) for an acceptable ceiling concentration and 50 ppm for a maximum peak during an 8-hour shift if no other measurable exposure occurs. The National Institutes of Occupational Safety and Health (NIOSH) established an H_2S exposure level of 10 ppm (10 minutes) as a maximum permissible limit (once per 8-hour shift), with continuous monitoring required where H_2S concentrations could be 50 ppm or greater (31). OSHA is currently revising their standards in a comprehensive guideline document applicable to confined spaces.

H_2S is an explosive as well as toxic gas. The lower explosive level for H_2S is 4.3 percent by volume in air and the upper explosive limit is 45 percent by volume in air. When the H_2S concentration is within this range, a spark can cause an explosion. Table 2-10 summarizes the hazardous nature of H_2S.

Table 2-10. Hazardous Characteristics of Hydrogen Sulfide Gas

Chemical Formula: H_2S

General Properties:
- Irritant and poisonous volatile compound
- Rotten egg odor in small concentration
- Exposure for 1½ to 2 minutes at 0.01% impairs sense of smell
- Odor not evident at high concentrations
- Colorless
- Flammable

Specific Gravity (compared to air = 1.0): 1.19

Physiological Effects:
- Impairs sense of smell rapidly as concentration increases
- Death in few minutes at 0.2%
- Exposure to 0.07% to 0.1% rapidly causes acute poisoning
- Paralyzes respiratory center

Maximum Safe 15-Minute Exposure: 20 ppm (OSHA)

Explosive Range, percent by volume in air:
- Lower Explosive Limit: 4.3
- Upper Explosive Limit: 45.0

Likely Location of Highest Concentration:
- Near bottom of confined space, but may be higher if air is heated and highly humid
- Areas of turbulence in collection system
- Low-lying flat sewers

Most Common Source: Sewer gas or sludge gas resulting from wastewater or wastewater constituents that have undergone anaerobic decomposition

2.8.2 Safety Practices for Confined Spaces

The dangers and toxicity of H_2S are well documented. The importance of following proper entry procedures for confined spaces is now obvious and should never be overlooked. Monitoring the air can be accomplished economically in a short time. The cardinal rule for everyone planning to enter a confined area where H_2S may be present is "Never Trust Your Senses" (32). What may look like a harmless situation may indeed be a potential threat.

The most common atmospheric conditions that constitute hazards are:

- Oxygen deficiency
- Combustible gases or vapors
- Toxic gases and vapors

One should always anticipate that any one or a combination of the above atmospheric conditions might exist within a collection system or confined space within the wastewater treatment plant. Procedures should be developed by each agency involved in the collection and treatment of wastewater for atmospheric monitoring and testing. A safety program should include consideration of the proper ventilation, entry, and extraction of personnel. Proper training of personnel and maintenance of monitoring equipment are mandatory for an effective safety program.

2.8.3 Monitoring for H_2S

When testing underground structures for H_2S, it is important to remember that H_2S is heavier than air, and its presence will not likely be detected at ground level. Therefore, one must lower the monitoring device into underground structures such as manholes and test at different levels. There are several types of devices available to test for H_2S:

1. Electrochemical sensor
2. Metal oxide semiconductor
3. Colorimetric detector tube or badge
4. Photoionization
5. Solid-state sensitized film (with ceramic chip)

Some of these devices are portable, continuous monitors, such as metal oxide semiconductors or electrochemical sensors that have visible or audible alarms which alert the worker of an H_2S concentration of 10 ppm or the OSHA ceiling limit of 20 ppm (33). A 10-ppm or 20-ppm concentration can be used as a go/no-go signal for entry or exit of a confined space. These monitors range in price from about $650 to $1,200 (1984). They can also be supplied with digital indicators to display the concentration of H_2S gas within the working environment.

Another device is a badge that contains an H_2S-sensitive indicator that will change to a dark color at 5 ppm. These devices are available for about $30 per dozen and are considered go/no-go type warnings.

In addition, colorimetric indicator tube testers are marketed which will read a given concentration of the gas. These devices are equipped with a bellows-type pump and, when fitted with the proper tubes, can detect other possible toxic substances found in sewer systems, such as carbon monoxide. The pumps draw a calibrated volume of air through a detector tube. The detector tube contains a reagent (in the H_2S case, lead acetate) which changes color in the presence of the gas. The extent of the discoloration indicates the amount of toxic gas present in parts per million. These tube testers cost approximately $300 with 10 tubes costing an additional $30 (1984). The tubes are available in different scale readings and for long-term duration usage to determine time-weighted average values.

A method has been developed for sampling heavier-than-air gases. This method uses a J-tube arrangement to sample near the water surface. A float is placed on the bend in the "J" to suspend the monitoring device above the water level. A color-sensitive indicator badge or an indicator tube with hose connection can be attached to the tube so that a representative sample can be taken as close as possible to the water surface. This will indicate if entry is safe or if further monitoring is required.

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When an NTIS number is cited in a reference, that reference is available from:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650

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Chapter 3

Odor and Corrosion Control In Existing Wastewater Collection Systems

3.1 Introduction

This chapter presents alternatives for control of odor and corrosion in existing wastewater collection systems. Since odor and corrosion problems are typically related to the presence of H_2S, the alternatives are aimed at preventing sulfide generation or removing sulfides through chemical or biological action. In-stream sulfide control alternatives fall into two major categories: 1) improving the oxygen balance (operation and maintenance, air injection or entrainment, oxygen injection); and 2) chemical addition (chemical oxidation, inhibition of sulfate reduction, precipitation, pH control).

3.2 Improving the Oxygen Balance

As previously discussed, presence of greater than 1.0 mg/l DO in the wastewater stream is sufficient to prevent sulfide buildup, since any sulfide that might be produced in the slime layer will be aerobically oxidized to thiosulfate. With no dissolved sulfides present in the stream, H_2S emission to the sewer atmosphere will not occur. Several techniques are available for improving the oxygen balance in wastewater collection systems, as discussed in the following sections.

3.2.1 Operation and Maintenance

Proper operation and maintenance of a sewer system can minimize unnecessary oxygen depletion. Partial blockages in the sewer can cause a backup of flow, resulting in lower velocities and deposition and accumulation of organic solids and debris. Such conditions favor the reduction of sulfate to sulfide, and the subsequent release of H_2S to the sewer atmosphere. These conditions can be minimized, however, by instituting a regular program of sewer inspection and cleaning. Inspections can be made manually or remotely, for example, by using television cameras. Several different techniques are available for sewer cleaning, including hydraulic methods and mechanical systems. Regular cleaning has been shown to temporarily reduce the rate of sulfide buildup, particularly where deposition of organic solids is a problem (1).

Since substantial sulfide generation can occur in a short time period in sewer lines used for flow equalization, constant-speed pump stations should be operated with start-stop cycles that are short enough to avoid backup of wastewater into influent lines and to avoid excessive wet-well detention times.

The effect of sluggish sewer flows, including increased depth of flow and prolonged surcharged conditions, can increase sulfide buildup due to decreased reaeration opportunity. However, shorter duration surcharged conditions caused by infiltration/inflow are often characterized by weaker wastewater and colder temperatures that may result in reduced sulfide generation due to higher flow velocities, scouring of accumulated solids, and reduced biological activity.

3.2.2 Air Injection

Improvement in the oxygen balance in collection systems can be realized by the addition of air into the flowing wastewater. Addition of sufficient DO can prevent or significantly reduce further sulfide generation from occurring and allow biochemical oxidation of existing dissolved sulfides. Methods of air addition include:

1. Direct injection of compressed air into force mains
2. Use of Venturi aspirators in force mains or lift stations
3. Use of air lift pumps at lift stations
4. U-tube dissolution using either compressed air or Venturi aspirators in either gravity lines or force mains
5. Pressure tank air dissolution systems for gravity lines.

3.2.2.1 Air Injection Alternatives

a. Direct Injection of Compressed Air

Direct injection of compressed air into force mains has been practiced at several locations in the United States. Increased pressure in a force main allows greater dissolution of oxygen into the stream. At atmospheric pressure and 21°C (70°F), water will dissolve approximately 2 percent air by volume; this increases to 4 percent at 103 kPa (15 psig), and 6 percent at 207 kPa (30 psig).

Direct injection of compressed air into force mains has been studied in California (3)(7), Texas (2)(5), Louisiana (4), and New Jersey (6). A typical direct air injection system is shown in Figure 3-1 and data from selected studies of compressed air injection into force mains for sulfide control are summarized in Table 3-1. Note the wide variation in air injection rates necessary to achieve sulfide control. Required air injection rates vary depending on wastewater characteristics (oxygen uptake rate), detention time in the force main, temperature and pressure of the system, hydraulic profile of the force main, and degree of sulfide control desired. "Rules of thumb" suggest providing air at the rate of 0.75 to 2.25 m^3/m^3 (0.1 to 0.3 standard cu ft/gal) of wastewater or providing an air flow of 0.7 to 1.3 $m^3/hr/cm$ (1 to 2 cfm/in) of pipe diameter (5). In practice, full characterization of the wastewater and information on hydraulic characteristics of the pipe are necessary before air requirements can be accurately estimated.

Figure 3-1. Typical system for injecting air into force main.

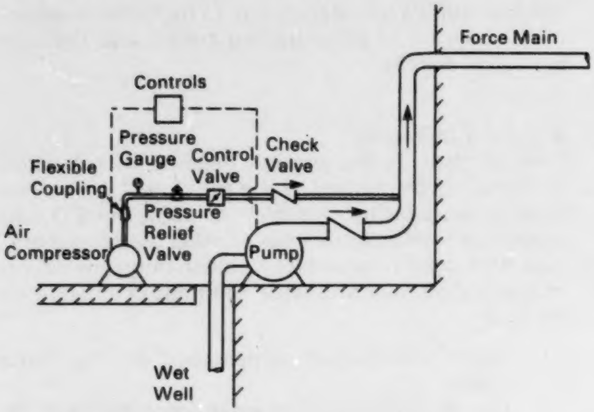


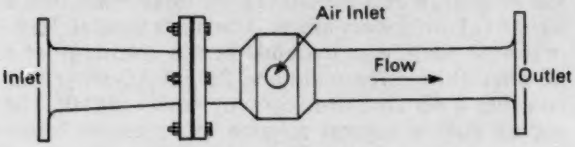
Table 3-1. Selected Studies of Direct Injection of Compressed Air into Force Mains for Sulfide Control

			Total Sulfide	
Diam.	Length	Air Input	Pump Station	Discharge of Force Main
cm	m	m^3/d	mg/l	
Los Angeles, CA (3)				
15	635	0	Avg.	0.32
		16.3	0.2	0.06
		32.6		0.06
15	340	0		0.20
		24.5	Avg.	0.31
		36.7	0.1	0.78
		53.0		Trace
		53.0		0.05
		53.0		0.10
25	525	0		16.4
		53.0		12.4
		61.2		14.8
		61.2		12.9
		61.2	Avg.	12.3
		61.2	2.1	3.4
		65.2		3.2
		73.4		8.0
		77.5		7.5
		81.6		4.5
30	455	0		4.6
		139	Avg.	0.2
		171	0.7	0.2
30	600	0		5.7
		57.1		5.6
		89.7		3.8
		110	Avg.	1.9
		163	0.5	3.5
		428		0.25
		428		0.2
		428		0.25
Port Arthur, TX (2)				
41	1,170	0		2.3
		489		0.7
		979		2.1
		1,350	Avg.	2.7
		1,470	1.4	0.6
		1,960		0.3
		2,450		0.1
		2,690		1.5
		4,078		0.1
76	1,340	0	Avg.	5.6
		4,490	2.3	1.4
Trinity River Authority, TX (5)				
76	1,550	0	Range	5 to 8
		3,260	0 to 3	<0.3
Sacramento, CA (7)				
60	1,700	0		0.45
		410	Avg.	0.39
		560	0.3	0.26
		860 to 920		0.19

b. Venturi Aspirators

Venturi-type aspirators have been employed for sulfide control in force mains and lift stations. A Venturi aspirator operates on the principle that liquid flowing at high velocity through a nozzle of decreasing diameter creates a negative pressure at the discharge side of the restriction. Provision of an opening to the atmosphere at this point allows air to be drawn into the device. Turbulence at the discharge provides for intimate mixing of air and water. A typical Venturi aspirator is shown in Figure 3-2.

Figure 3-2. Typical Venturi aspirator.



A Venturi aspirator was installed in a 20-cm (8-in) force main in Jefferson Parish, Louisiana as part of a demonstration project on sulfide control. This device aspirated approximately 200 to 245 m^3/d (5 to 6 cfm) of air at a wastewater flow of 5,440 m^3/d (1,000 gpm), resulting in an air-water ratio of about 4 percent (4).

One proprietary system that has been employed for sulfide control in lift stations draws wastewater from the wet well, pumps it at a pressure of 138 kPa (20 psi) through the Venturi aspirator, and recirculates the aerated sewage back to an upstream manhole (8)(9). Depending on the magnitude of the sulfide problem, recirculation ratios can be varied. Typical recirculation ratios vary from 0.75:1 to 1:1. This system is depicted in Figure 3-3.

c. Air Lift Pump

An air lift pump was installed in a 75-cm (30-in) diameter gravity sewer in Jefferson Parish, Louisiana to control sulfide by upstream aeration (4). It was installed because of its ability to provide simultaneous aeration and pumping. A diagram of the air lift pump used in Jefferson Parish is shown in Figure 3-4.

Several different aeration techniques were studied at various points in the Jefferson Parish collection system. At the location of the air lift pump, 60 percent of the entering wastewater was unaerated, while 40 percent was subjected to upstream aeration by one or more alternative techniques. Prior to any aeration, dissolved sulfides averaged 0.63 mg/l at a point 255 m (835 ft) downstream of the air lift pump. After system aeration was begun, dissolved sulfides at the sampling point were reduced to zero (4).

d. U-Tube Aeration

The use of U-tubes for in-stream aeration to control sulfides in collection systems has been evaluated at Jefferson Parish, Louisiana (two Venturi-aspirated U-tubes) and Port Arthur, Texas (two parallel compressed air U-tubes) (2)(4)(10). In addition, U-tube aeration was investigated at Sacramento, California (7).

A diagram of a typical U-tube aerator is shown in Figure 3-5. Air (or oxygen) is introduced at the top of the downleg, from which the air-sewage mixture flows downward through an expanded pipe section. This allows reduced velocity and greater residence

Figure 3-3. Application of Venturi aspirator for sulfide control in wet well.

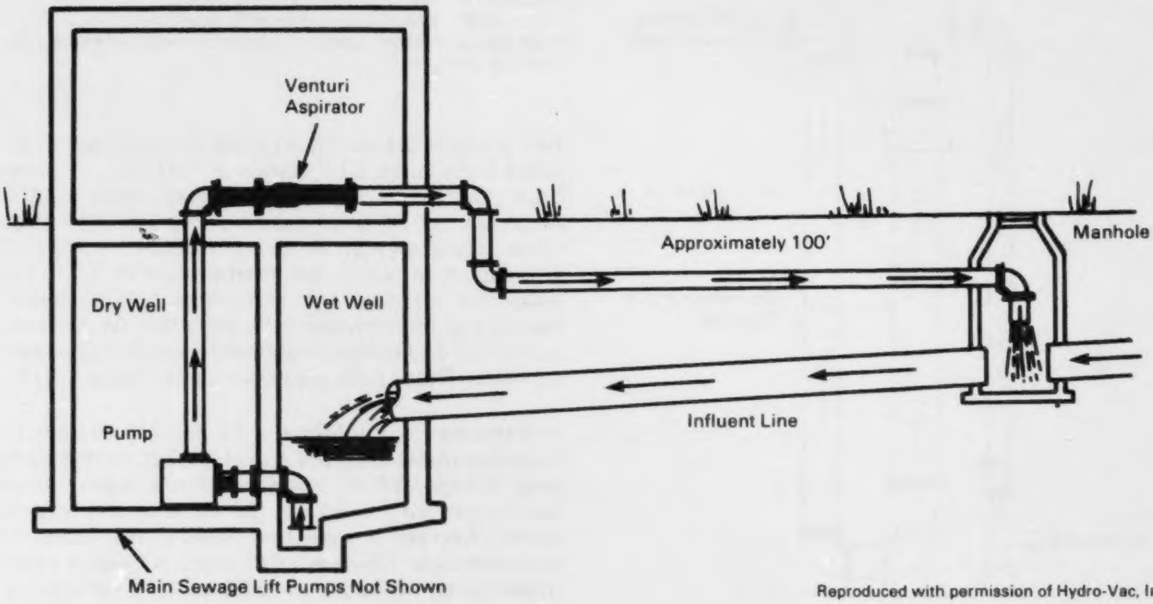


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Figure 3-4. **Air lift aerator installed at Jefferson Parish, LA.**

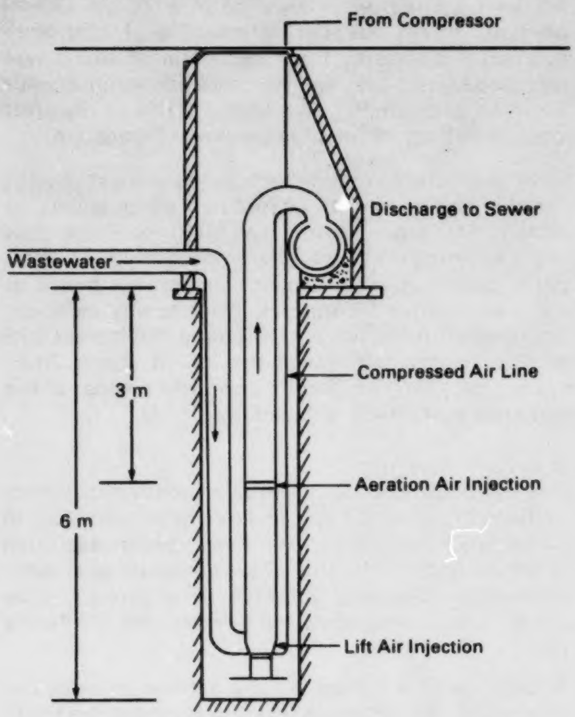
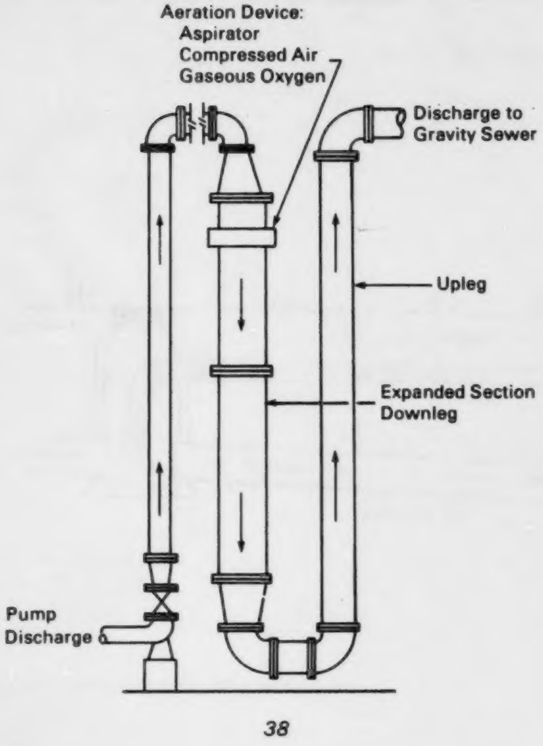


Figure 3-5. **Typical U-tube installation (2).**



time to promote oxygen transfer under conditions of increasing hydrostatic pressure. The oxygen enriched wastewater then continues through an upleg of reduced diameter to increase the velocity and prevent deposition of solids.

The larger capacity, 12.8-m (42-ft) deep Venturi-aspirated U-tube at Jefferson Parish was installed at the discharge of a 30-cm (12-in) force main into a 53-cm (21-in) gravity sewer. The other U-tube, 16.5-m (54-ft) deep, was installed at the discharge of a 25-cm (10-in) force main and 25-cm (10-in) gravity line into a 45-cm (18-in) gravity sewer (4)(10). The overall sulfide control scheme at Jefferson Parish actually employed four aeration devices located at various points in the collection system: two Venturi-aspirated U-tubes, an air lift pump, and an in-line Venturi aerator. Data were collected at numerous sample stations before and after simultaneous operation of all aeration devices. This made evaluation of the performance of each individual device difficult. Data in Table 3-2 were taken from samples collected at downstream stations which best represent the performance of the air aspirated U-tubes (4).

Table 3-2. **Performance of Aspirated Air U-Tubes at Jefferson Parish, LA (4)**

Sample Location[a]	Wastewater Flow	Air:Water Ratio (Volume)	Dissolved Sulfides Before Aeration	Dissolved Sulfides After Aeration[b]
	m³/d		mg/l	
A	9,267	0.08	0.51	0.05
B	3,543	0.05	0.42	0.11

[a] Location A: 113 m downstream of U-tube 1
Location B: 122 m downstream of U-tube 2

[b] Peformance reflects partial contribution from other aeration devices upstream.

Two parallel 6.1-m (20-ft) deep compressed air U-tubes installed at a lift station in Port Arthur, Texas discharged into a 75-cm (30-in) gravity sewer (2)(10). Although the intended design of the Port Arthur U-tubes provided for an air flow of 1,630 m^3/d (40 cfm) to yield an air:water volumetric ratio of 0.21, the maximum air flow the compressor could deliver resulted in an air:water ratio of 0.082. As a result, only minor reductions in sulfide concentrations were achieved. These data are presented in Table 3-3 (2).

In Sacramento, California, a 12.2-m (40-ft) deep U-tube was installed at the end of a 1,720-m (5,650-ft) long 60-cm (24-in) diameter force main which discharged into a 69-cm (27-in) diameter gravity sewer. Average dry weather flows at the site were approximately 7,600 m^3/d (2 mgd). Air:water ratios investigated ranged from 0.06 to 0.16 by volume.

Table 3-3. Performance of Compressed Air U-Tubes at Port Arthur, TX (2)

Wastewater Flow	Air:Water Ratio (Volume)	Sulfide Concentrations In (Pump Discharge) Total	Sulfide Concentrations In (Pump Discharge) Dissolved	Sulfide Concentrations Out (U-tube Discharge) Total	Sulfide Concentrations Out (U-tube Discharge) Dissolved
m^3/d		mg/l			
7,179	0.034	4.3	4.0	2.6	2.4
7,190	0.045	5.3	4.2	2.7	2.3
7,223	0.045	1.7	1.6	0.9	0.6
7,086	0.057	2.3	2.1	1.5	1.1
7,163	0.063	5.0	4.8	2.8	2.3
6,890	0.071	1.8	1.5	1.5	1.1
7,086	0.081	4.2	3.8	3.8	3.0
6,945	0.082	2.4	2.0	1.7	1.2

Results from this study are shown in Table 3-4 (7). It was found that variation in air:water ratios over the ranges studied had little observable effect on the degree of sulfide reduction. However, increased air:water ratios did result in increased DO levels downstream. At air:water ratios above 0.15, dispersion of the air was so poor that air blocks occurred in the downleg above the air injection collar.

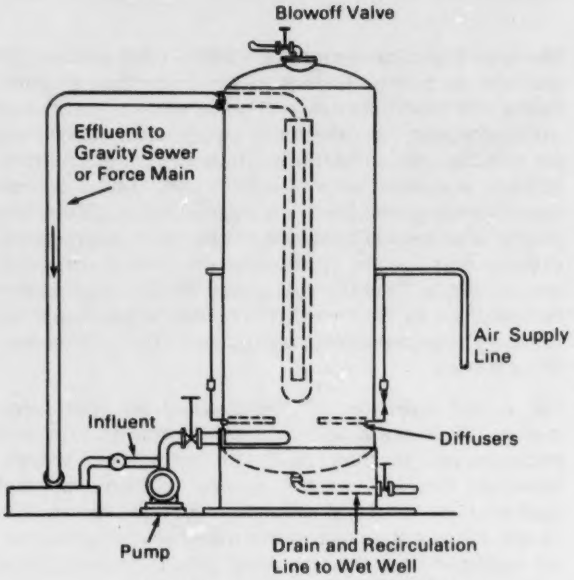
Table 3-4. Performance of Compressed Air U-Tubes at Sacramento, CA (7)

Wastewater Flow	Air:Water Ratio (Volume)	Total Sulfide Concentration In	Total Sulfide Concentration Out	Total Sulfide Concentration 1,000 m Downstream
m^3/d		mg/l		
8,740	0	–	–	0.60
9,050	0.06-0.08	0.61	0.46	0.17
8,740	0.11-0.13	0.76	0.52	0.17
8,630	0.15-0.16	0.69	0.56	0.13

e. Pressure Tank Air Injection

Pressure tank air injection is a technique that takes advantage of the increasing solubility of oxygen with increasing pressure. Such a system is applicable to sulfide control at lift stations discharging to gravity sewers or in short force mains where detention or contact time does not allow adequate oxygen dissolution and sulfide oxidation. Pressure tank air dissolution systems were installed and evaluated at two lift stations at Port Arthur, Texas (2). A diagram of a typical system is shown in Figure 3-6. After problems were noted with deposition of solids in the pressure vessel, a recirculation line from the bottom of the pressure tank to the lift station wet well was installed.

Figure 3-6. Pressure tank dissolution system (2).


Blowoff Valve
Effluent to Gravity Sewer or Force Main
Air Supply Line
Influent
Diffusers
Pump
Drain and Recirculation Line to Wet Well

The use of pressurized tank injection systems at Port Arthur resulted in air:water volumetric ratios considerably higher than with other air injection methods evaluated. Air:water ratios were 3.9 for one installation and 3.5 for the other. These ratios were considered excessive for adequate sulfide control. Performance data for the two pressurized tank air injection systems are summarized in Table 3-5 (2).

3.2.2.2 Air Injection Equipment

Equipment requirements for an air injection system vary with the type of system selected. For applications requiring small quantities of air, less than 14 m^3/min (500 cfm), equipment requirements may be minimal and may consist, for example, of merely a pump and a Venturi aspirator. Larger applications will probably require multiple compressors, more sophisticated control systems, and possibly separate dissolution systems.

Table 3-5. Performance of Pressure Tank Air Injection at Port Arthur, TX (2)

Location	Average Sulfide Concentration		
	Total	Dissolved	DO
	mg/l		mg/l
A:			
Upstream manhole	2.3	2.0	0
Wet Well	0.6	0.5	2.1
Pressure tank effluent	0.3	0.2	6.3
Force main discharge (180 m)	0.1	0	4.8
B:			
Wet Well	3.9	3.1	0
Pressure tank influent	1.1	0.9	0.1
Pressure tank effluent	0.1	0	4.9
Force main discharge (30 m)	0.2	0.05	3.9

Many air injection systems are installed in existing lift stations or pump stations in the collection system. Some site modifications will generally be required to accommodate the additional equipment, depending on specific site conditions. Such modifications may include provision of a concrete pad, paved access roads, fencing, and electrical supply, although electric power is generally available to operate compressors, pumps, and control systems at the site. If sufficient area is available within the pump station, equipment can often be located inside the facility. If not, separate housing for compressors and associated controls may be required.

For *direct injection of compressed air* into force mains, hardware would consist of compressor(s) and receiver or reservoir tank, air piping and valves, injection nozzles, and a control system. Control options are discussed in Section 3.2.2.3. Air-cooled, single-stage compressors are generally recommended because they develop adequate capacity at low pressure (11). Compressors should be designed for continuous duty and be sized to handle peak aeration demands with a reasonable duty cycle. Most standard compressors are rated at 690 kPa (100 psig). At lower pressures, more air can be delivered at reduced power. Piping not in contact with wastewater should be constructed of standard weight galvanized steel (up to 15-cm diameter) or galvanized spiral steel (20 to 60 cm). For submerged piping, the following materials are recommended: less than 7.5-cm (3.0-in) diameter—standard weight galvanized steel, painted outside; greater than 7.5-cm (3.0-in) diameter—cast iron, galvanized steel (12). Several different nozzles and diffusers have been used in force main aeration. If the point of injection is at the bottom of a vertical riser, diffusers may enhance dissolution. However, for typical applications of air injection into a sloping force main, air bubbles coalesce into large bubbles within a short distance downstream, regardless of the manner of air injection. For these applications, the usual practice is to merely pipe the air into a suitable connection in the main.

For *air injection by Venturi aerators*, the major equipment required is the aeration device and a pump. Most Venturi aerators require a minimum pressure at the inlet and a minimum wastewater flow rate to achieve the desired rate of air aspiration. Bypass valves and piping are recommended to allow diversion of flow. Alternatively, the aspirator and pump can be located in a recirculation line that allows the flow to bypass the device when the pump is not operating.

Air lift pumps can be used to provide both pumping and aeration. Equipment required for an air lift pump consists of a compressor, air feed piping, and the air lift device. Af Jefferson Parish, Louisiana, "lift air" was injected at the bottom of the 6-m (20-ft) riser, and "aeration air" was added at a point approximately 3-m (10-ft) deep in the airlift casing (4).

U-tube dissolution systems employ either compressors or Venturi aerators for air supply. U-tubes can be of any depth, but little advantage is gained at depths exceeding 30 m (100 ft), as total cost per unit of oxygen dissolved increases due to the need for specialized drilling equipment (13). U-tubes may be fabricated from steel or fiberglass-reinforced plastic pipe and secured within standard steel well casing. To avoid plugging, a pair of vertical pipes connected by a 180-degree return bend is recommended. Use of concentric pipes or a rectangular trough with center divider may be less expensive, but such configurations are more subject to plugging when used for raw wastewater aeration (10). If sufficient head is not available, pumps may be required to force the air-water mixture through the U-tube. With aspirated air systems, little control over air flow can be exercised, the amount of air aspirated being a function of the aspirator design, inlet pressure, and wastewater flow rate. For compressed air U-tubes, air injection rates can be regulated to some degree by programmed control of the compressor to inject air only when required on an intermittent basis. At Port Arthur, Texas, compressed air was introduced into an air injection collar located near the top of the downleg. Thirty-two 0.3-cm (0.12-in) holes were spaced equally around the circumference of the line through which compressed air was introduced (10).

Equipment requirements for a *pressure tank dissolution system* consist of an air compressor, pump, pressure vessel, air piping and diffusers, pressure relief valve (piped to discharge line), and air flowmeters and valves. Wastewater bypass piping and valves should also be provided. Provision of periodic or continuous recycle from the bottom of the vessel to an upstream location may be necessary to control solids accumulation due to deposition.

3.2.2.3 Design of Air Injection Systems

a. Air Requirements

The first step in design of an air injection system is to estimate air requirements to achieve the desired objectives of sulfide control. This is based largely on expected oxygen uptake rates in the sewer. The overall oxygen balance in a sewer can be described by the following equation (3):

$$\frac{dO_2}{dt} = R_f - R_r - R_s \qquad (3\text{-}1)$$

where,

$\frac{dO_2}{dt}$ = rate of change of oxygen concentration as the stream moves down the sewer, mg/l-hr

R_f = rate of gain of oxygen by surface reaeration, mg/l-hr

R_r = rate of loss of oxygen due to reaction in the stream (oxygen uptake rate of wastewater), mg/l-hr

R_s = rate of loss of oxygen due to reaction on the slime layer, mg/l-hr

In force mains, the surface reaeration rate, R_f, can be assumed to be zero. In gravity sewers, R_f can be estimated using equation 2-14. Values for R_r, oxygen uptake in the wastewater, should be experimentally derived from sampling at several locations along the sewer reaches of interest. The sampling period should fully cover normal diurnal flow variations. R_s can be estimated using Equation 2-8, as experimental determination is difficult and time-consuming.

Several different approaches can be employed to estimate air requirements for sulfide control in a particular sewer reach. One approach involves the following procedures:

1. Determine typical initial oxygen uptake rates, R_r, in wastewater from sampling programs and develop oxygen uptake curve, as shown in Figure 3-7.
2. Calculate wastewater detention time in the sewer reach at minimum, average, and peak flows.
3. Measure oxygen uptake rate at 1-hour increments on samples collected during periods of maximum oxygen uptake.
4. Plot oxygen uptake rate with time, as shown in Figure 3-8.
5. Calculate area under each curve in Figure 3-8 for cumulative increments of time to yield oxygen demand in mg/l.
6. Plot ratios of oxygen demand to initial oxygen uptake rate, as shown in Figure 3-9.

Figure 3-7. Example of determination of oxygen uptake rate.

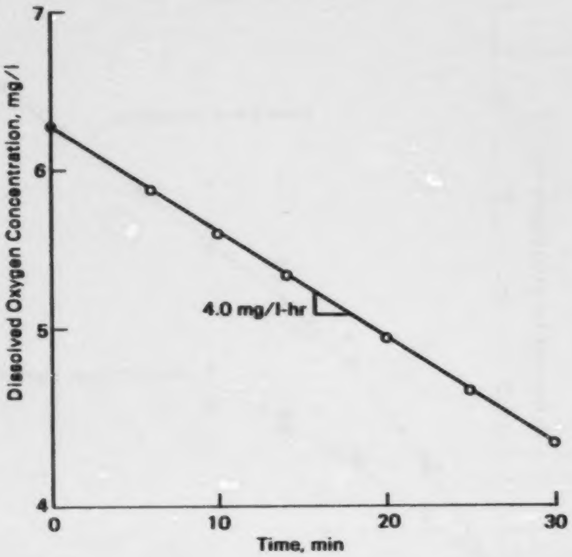
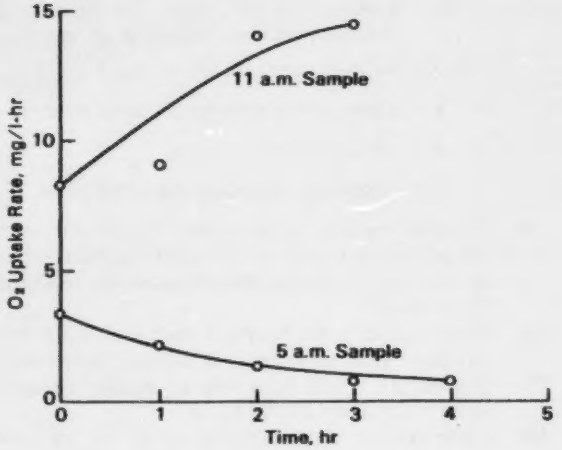


Figure 3-8. Example of variation in oxygen uptake rate with time.

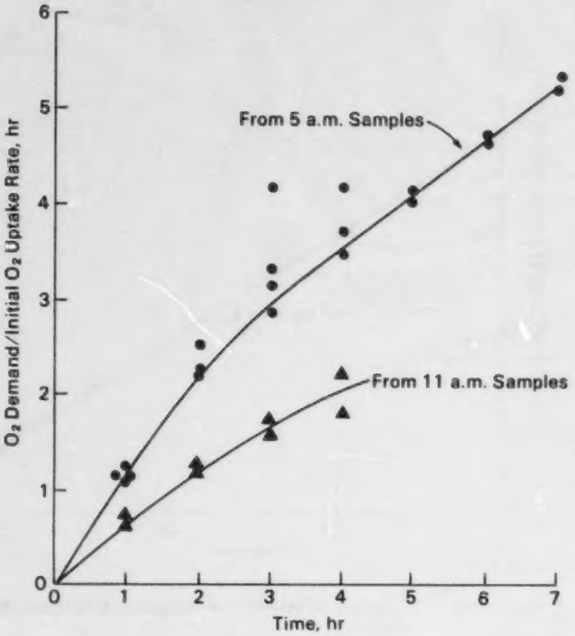


7. Multiply design oxygen uptake rate by above ratios at detention times corresponding to minimum, average, and daily peak flow to yield oxygen requirements in mg/l. For design purposes, use ratios from curve (Figure 3-9) which represents the greatest potential for oxygen usage.
8. Calculate oxygen uptake rate by slime layer, R_s:

$$R_s = 5.3[O_2](su)^{1/2}R^{-1} \qquad (2\text{-}8)$$

where,

Figure 3-9. **Example ratio of cumulative oxygen demand to initial oxygen uptake vs. time.**



R_e = loss of oxygen from the stream by reaction with the slime layer, mg/l-hr

$[O_2]$ = oxygen concentration, mg/l

s = slope of the energy grade line, m/m

u = stream velocity, m/s

R = hydraulic radius of the stream, m

9. Multiply oxygen uptake rate in slime layer by ratios obtained in Step 7 to yield oxygen required by slime layer (mg/l) at various detention times (flow rates).
10. Add results from Steps 7 and 9 to yield total oxygen required in mg/l at various flow rates.
11. Determine mass flow rate of oxygen required (flow x concentration), kg/d.
12. Calculate air requirements in m^3/hr for minimum, average, and maximum flows.

For direct air injection into force mains, an alternative approach is to use the following equation (14):

$$Q_a = \frac{(6 \times 10^{-4})VR_r}{P_1 \log\left[(P_1 + P_2)/P_1\right]} \times \frac{u_a + u_w}{u_a} \qquad (3\text{-}2)$$

where,

Q_a = required air flow at ambient force main pressure, m^3/min

V = volume of pressure main, m^3

R_r = rate of loss of oxygen due to reaction in the stream (oxygen uptake rate of wastewater), mg/l-hr

P_1 = ambient atmospheric pressure, atm

P_2 = gage pressure at injection point, atm

u_a = velocity of the air relative to the wastewater, m/s = 1.66 $d^{1/2}$, where d = pipe diameter, m

u_w = velocity of the wastewater, m/s

This equation generally predicts more than enough air for sulfide control, since it is based on maintaining an excess of oxygen. In actual practice, adequate sulfide control can often be achieved using one-third or one-half as much air as would be required to satisfy the oxygen-consuming capability of the wastewater. It should also be noted that Equation 3-2 is not useful at pressures greater than 276 kPa (40 psi) due to the large impact on both oxygen and nitrogen dissolution.

This equation also requires knowledge of the oxygen reaction or uptake rate, R_r, of the wastewater. This should be determined through sampling and analysis. If this cannot be done, Table 3-6 can be used as a general guide to selecting oxygen reaction rates.

Table 3-6. **Suggested Oxygen Reaction Rates (15)**

Age of Wastewater	Suggested R_r (conservative) mg/l-hr
1 hr @ 20°C or ½ hr @ 30°C	5
2 hr @ 20°C or 1 hr @ 30°C	10
Over 3 hr @ 20°C or 1½ hr @ 30°C	15

b. Control Systems

Since air requirements vary with diurnal flows and oxygen reaction rates, it is often desirable to program air injection rates to more closely match air requirements, thereby avoiding excess air use and associated higher expenditures of energy. Several different control approaches with varying levels of sophistication are possible. For single compressor installations, the compressor controls can be interlocked with pump starter circuits. A time-delay circuit may be employed to delay air injection until after the pumps have started in order to avoid gas locking. A common technique is to use a pressure switch on the receiver tank, which operates a device that lifts the air intake valves so that the compressor idles (6)(7). Although an idling compressor still utilizes energy, it is at a reduced rate. Air injection at a pump station in Gloucester County, New Jersey was controlled with a pressure switch on the receiver using the following strategy (6):

Condition	Response
Pressure < low pressure switch setting	Compressor operates
Pressure > high pressure switch setting	
1. Pump on	Compressor continues to operate
2. Pump off	Compressor idles for minimum run time, then shuts down unless pressure switch calls for compressor operation

Another simple control scheme is the use of two compressors of different capacities to provide three rates of air flow. For example, if diurnal air requirements varied between 10 and 30 m^3/hr, a 10-m^3/hr unit and a 20-m^3/hr unit could be specified, providing possible air flow rates of 10, 20, and 30 m^3/hr (5.9, 11.8, and 17.7 cfm). A timer circuit would actuate the compressors on a pre-programmed schedule based on diurnal variations in oxygen requirements.

A more sophisticated approach is the use of cam-operated controllers which actuate solenoid valves on metering lines. The lines are equipped with interchangeable flow-controlling orifices (16). Still another approach involves use of variable-speed compressor drives.

An important consideration in determining air requirements for sulfide control in a force main is the loss of air through air release valves. Air losses at each relief valve can be as much as 30 percent, depending on force main pressure and actuation pressures of the valve. Accounting for such losses may yield significantly higher design air flow rates than calculations based on oxygen uptake would indicate (16).

Location of the air injection point is critical in designing a direct air injection system. Maximum dissolution occurs in the vertical riser near the discharge of the pump, since this is the point of maximum velocity, pressure, and turbulence. In some cases, as with submersible pumps, this may not be practical, and alternative injection locations must be considered. Selection of an alternative injection point may, however, result in reduced dissolution efficiencies.

Evaluation of the plan and profile of the force main, location of air release valves, and materials of construction is necessary in the design of injection facilities to avoid accumulation of large gas pockets that can result in gas locks in the line or in pumps.

When injecting air into a force main, the amount of air fed into the stream will often exceed the solubility. As a result, gas pockets will form. This is not necessarily detrimental, as the gas pockets will be carried with the flow and will dissolve further downstream in the main, primarily in descending legs. However, in mains with low pump-head pressure and undulating profiles, gas pockets can form at high points between pump cycles, and may result in greater resistance to flow.

Gas locking of pumps can also occur, particularly where a downhill gradient exists immediately following the discharge riser from the pumps. When the pumps are operating, undissolved gas will form a long pocket in the downhill portion of the force main if conditions are not favorable to complete dissolution. When the pumps and air injection are stopped, the pocket travels upgradient and collects in the discharge riser. If the gas pocket is of sufficient volume and the check valves following the pumps are not watertight, gas may leak back into the pump to cause gas locking (17).

Operation of pumps while dry causes packing gland failure due to overheating. Problems associated with gas locking of pumps can be avoided by following these guidelines (17):

1. Avoid down grades immediately after the discharge riser.
2. Inject air after the down grade.
3. Control the size of the gas pocket by more efficient dissolution techniques.
4. Check static head to determine if it is sufficient to push air pocket back to pumps, and check volume of gas pocket.

Location of gas pockets in the discharge riser can be monitored by installing a "boiler glass" on the riser. The level of the gas pocket in the sight glass should never be more than one pipe diameter below the crown of the horizontal section of the main.

Where proprietary equipment is to be used for aeration, as in the case of Venturi aspirators and air lift pumps, manufacturers' representatives should be contacted to assist in equipment specification. Venturi aspirators have air capacity ratings based on a specific fluid flow rate and inlet pressure. Deviations from these conditions will result in fluctuations in quantities of air delivered by the device. This should be considered during design.

Air lift pumps are low in efficiency as pumps, but since they can serve as aeration devices, the combined efficiency is improved. At the Jefferson Parish, Louisiana, installation, the air lift was designed to raise the wastewater above the top of the sewer and let it fall back into the sewer. However, objectionable odors resulted from exhalation of air from the sewer.

This can be avoided by allowing the compressor to draw air from the sewer atmosphere. H_2S should not have any effect on the compressor because the temperature is high enough to prevent condensation, but droplets of moisture must be removed from the inlet air.

c. U-Tube Dissolution Systems

Design of U-tubes can be somewhat difficult due to the number of design and operating variables. A computer program has been used to allow optimization of design through parametric calculations (10).

Air entrainment can be accomplished using an aspirator device or a compressor and injector. Use of a compressor allows greater flexibility in feed rates, permits use of higher air-water ratios, and results in lower head losses than air aspirated systems. However, the Venturi aspirator requires little or no maintenance and requires no power other than that required to overcome head losses.

The U-tube flow configuration can take many forms, including: 1) a pair of vertical pipes connected by a 180-degree return bend; 2) a pair of concentric or eccentric pipes; 3) a single pipe with a flat, vertical partition; and 4) a rectangular trough with a vertical partition to separate the downward and upward flow passages. Concerns with pipe plugging or fouling have led to the use of two pipes connected by a return bend. For large U-tubes, two concentric pipes may be used if the cross-sectional areas of flow are sufficiently large to allow passage of solids.

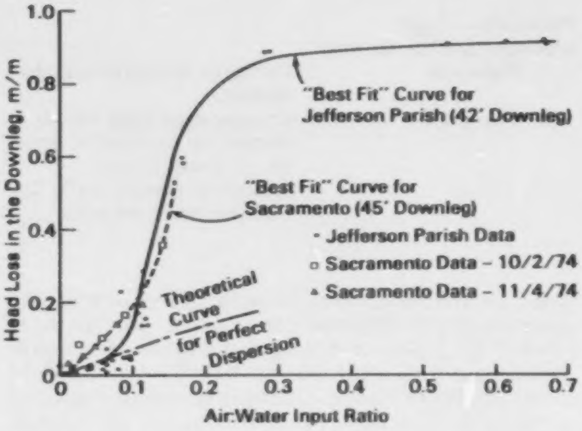
Cross-sectional areas of the downleg are greater than the upleg to allow greater detention time for oxygen transfer to occur in the downleg, and to prevent deposition of solids by increased velocity in the upleg. Typical velocities in the downleg are about 0.5 m (1.5 ft)/s, while velocities in the upleg should reach 1.2 m (4.0 ft)/s at least once a day to prevent accumulation of solids.

When the inlet pipe is raised above the outlet from the U-tube, relatively little change in performance can be expected, but the head loss is reduced. Raising the inlet pipe may be desirable for ease of construction or operation.

Head loss is an important consideration in the design of U-tubes. Head loss in an operating U-tube varies primarily with the air:water ratio, head loss increasing with increasing air injection rates (18). Figure 3-10 shows the relationship of head loss in the downleg to air-water ratio for the Jefferson Parish, Louisiana, and Sacramento, California, U-tubes (7). Unit head loss increases dramatically at air:water ratios between 0.1 and 0.3.

Location of the air injection point should be near the top of the downleg. This provides maximum contact time with the wastewater to allow dissolution to occur. Lowering of the injection point will reduce head loss through the system but at the expense of lower DO concentrations at the outlet.

Figure 3-10. **Head loss in downleg of U-tubes at Jefferson Parish, LA and Sacramento, CA (7).**



Figures 3-11 and 3-12 show typical design curves for compressed air U-tubes and aspirated air U-tubes, respectively. These curves were generated using a computer design program for U-tube aerators (10). For the specified flow rate and pipe sizes, these curves show expected outlet DO and head loss through the system as a function of oxygen supplied.

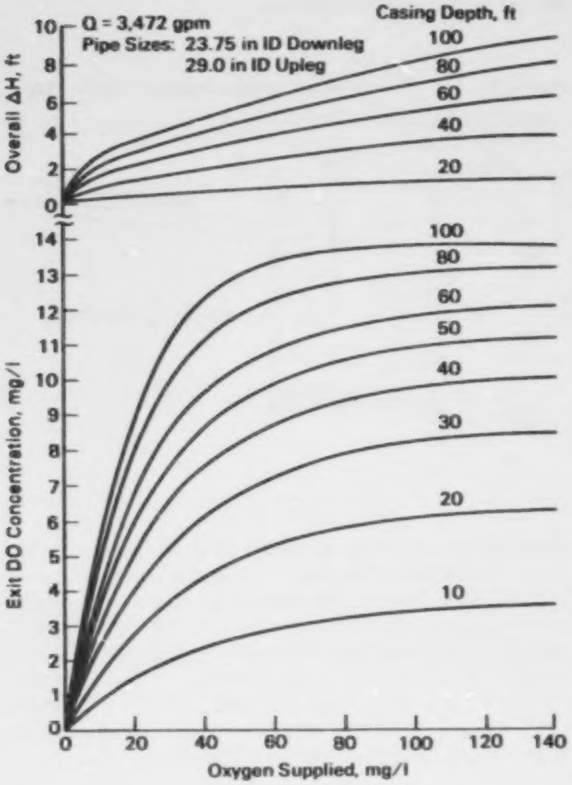
d. Pressure Tank Dissolution Systems

Design criteria for pressure tank air dissolution systems are not well established due to limited experience with the application of this technique for sulfide control. At Port Arthur, Texas, the volumetric air-to-water ratio was 3.91 at nominal conditions, corresponding to an oxygen application of 949 mg/l. At design conditions, detention time in the pressure tank was 17 minutes, and operating pressures were between 1 and 2 atmospheres. The design of the device required continuous operation of wastewater pumps and air compressors. As a result, maintenance and power costs were estimated to be higher than with other aeration alternatives (2).

3.2.2.4 Cost of Air Injection Systems

Typical costs of systems for direct injection of compressed air into a force main are shown in Table 3-7. These costs are budget level estimates (+30 percent, -15 percent). If a separate dissolution system is required, as for gravity sewer applications, additional capital costs would be incurred. Due to lack of definitive cost information, as for pressure tank dissolution, and the wide variability in site-specific

Figure 3-11. Typical compressed-air U-tube parametric designs (10).



design criteria, as for U-tubes, these costs have not been included.

3.2.3. Oxygen Injection

Injection of pure oxygen into force mains and gravity sewers is a demonstrated technique for sulfide control. Over 100 installations have been completed in the United Kingdom and Australia, and about 10 in the United States (19). Pure oxygen addition has several advantages over air injection:

1. Oxygen is five times more soluble in water than is air
2. Smaller quantities of gas need to be injected, reducing the likelihood of gas pocket formation
3. Compressors are not required
4. Greater solubility results in improved pumping efficiency and oxidation of existing sulfides, and maintenance of a higher residual DO to prevent further sulfide generation.

The efficiency of oxygen injection is dependent on the energy expended in dissolving the gas, the temperature and pressure of the stream into which the oxygen

Figure 3-12. Typical aspirated-air U-tube parametric designs (10).

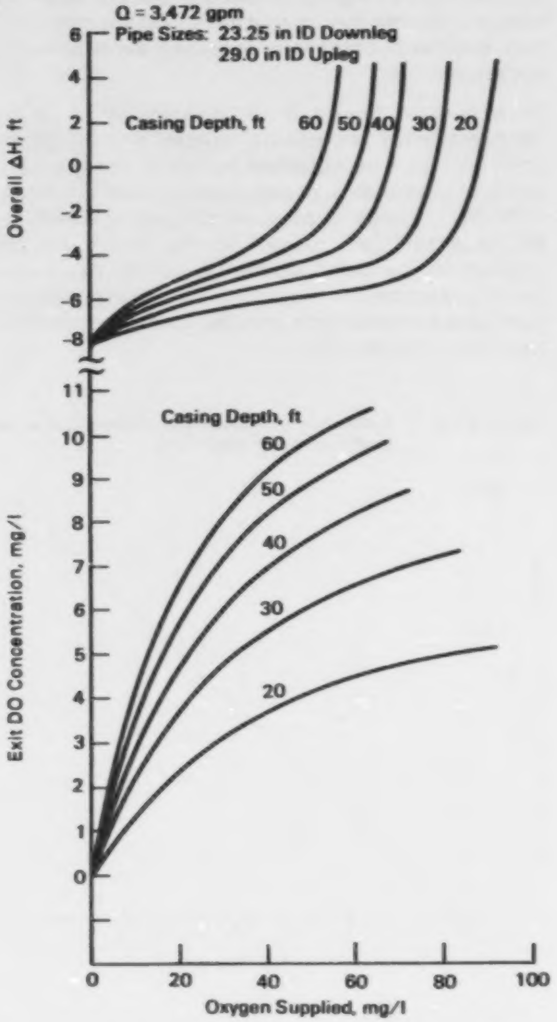


Table 3-7. Typical Costs for Direct Compressed Air Injection into a Force Main for H_2S Control (1984 $)

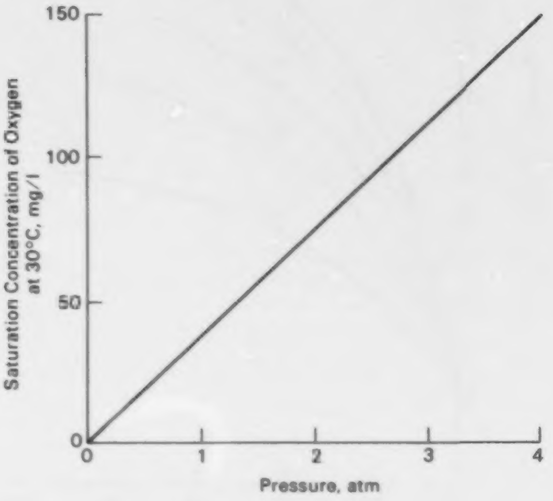
Condition	Capital Cost*
	$
Flow = 3,785 m^3/d Diameter = 25.4 cm Length = 1,600 m Pressure = 158 kPa Air flow = 2.5 m^3/min	21,000
Flow = 37,850 m^3/d Diameter = 61.0 cm Length = 1,600 m Pressure = 158 kPa Air flow = 15.3 m^3/min	54,000

*Includes concrete pad, compressor, piping, valves, start-stop controls, pre-fab building, and installation.

is introduced, and the method used for injection. Maximum oxygen dissolution is obtained in zones with sufficiently high turbulence to produce additional bubbles. Bubble size is also an important consideration. Smaller bubbles provide greater surface area for oxygen transfer.

As shown in Figure 3-13, dissolution of oxygen improves with increasing pressure (19). Higher pressures can be obtained by increased discharge heads of force mains, or use of pressurized sidestream systems, U-tubes, or other dissolution systems. It is desirable to inject oxygen at the lowest possible location in the pump discharge piping. Application points are located in the collection systems such that downstream conditions provide for maximum maintenance of residual DO.

Figure 3-13. **Saturation concentration of oxygen in water at different pressures (19).**

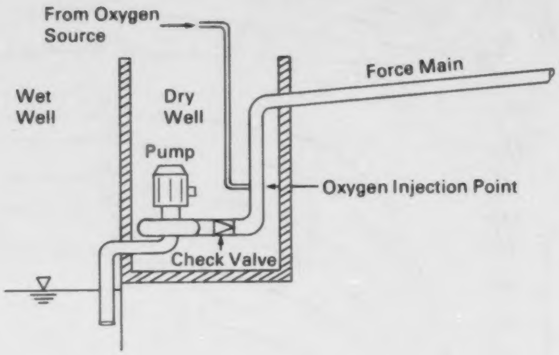


3.2.3.1 Oxygen Dissolution Alternatives

a. Direct Oxygen Injection Into Pipes Under Pressure
Direct injection of oxygen is the simplest and most common means of oxygen dissolution but is only applicable to force mains or pipes under pressure. In gravity systems, and in some force mains, insufficient pressure and turbulence exist to achieve desired dissolution, requiring consideration of alternatives to artificially create these conditions.

Figure 3-14 shows a direct oxygen injection system at a pump station. Oxygen is injected at a point of high turbulence, favoring the formation of small bubbles which will readily dissolve. Injection at the bottom of the vertical riser improves dissolution due to increased pressure and reduced coalescing of the bubbles. However, undissolved bubbles will tend to coalesce in the horizontal portions of the main, where a gas-liquid interface forms. Turbulent conditions downstream promote transfer of oxygen from the gaseous to the liquid phase.

Figure 3-14. **Typical direct oxygen injection system (19).**



Results from a study of direct oxygen injection into force mains at Port Arthur, Texas are summarized in Table 3-8 (2). These data show that direct injection of pure oxygen into force mains can be successfully used to control sulfides. Results of field tests of direct oxygen injection into a force main in Sacramento, California are presented in Table 3-9 (7). As can be seen, oxygen injection effectively controlled sulfides at the lowest oxygen dosage studied.

Siphons are often associated with sulfide generation due to long retention times during periods of low flow. Direct oxygen injection has been successful in controlling sulfides in three siphons of 99-cm (39-in) diameter in the Tyneside Sewage System in the United Kingdom (20).

The ideal location for oxygen injection into a siphon is the bottom elbow of the descending leg where high turbulence and greater pressures promote good oxygen transfer. For efficient dissolution, a minimum wastewater velocity of 0.6 m/s (2 ft/s) should be maintained. The lower pipe in a siphon must be horizontal or sloped upwards to prevent gas collection at the crown. The final ascending leg should also have vertical rise to promote gas-liquid intermixing.

b. Single U-Tube Injection
There have been several investigations of the single U-tube (Figure 3-5) for dissolving pure oxygen in wastewater (2)(10)(18). A U-tube aeration system will absorb pure oxygen with an efficiency of 60 to 80 percent when the volumetric rate of pure oxygen injection is 0.5 to 1 percent of the wastewater flow rate (18).

Table 3-8. Performance of Direct Oxygen Injection into Force Mains at Port Arthur, TX (2)

Force Main	O_2 Injection Rate	Sulfide Concentration			
		Wet Well		Force Main Discharge	
		Total	Dissolved	Total	Dissolved
	m³/d	mg/l			
Diameter = 40.6 cm	0	0.5	0.4	1.2	1.1
Length = 1,174 m	82	1.5	0.9	0.1	0
Flow = 6,540 m³/d	122	1.7	0.9	0.1	-
	163	1.0	0.7	0.2	0.1
	184	0.8	0.5	0.2	0
	204	1.9	1.3	0.1	0
	245	2.0	1.2	0.1	0
Diameter = 20.3 cm	0	2.3[a]	1.9	6.4	4.6
Length = 960 m	204	2.3[a]	1.9	0.9	0.5
Flow = 3,270 m³/d	245	2.3[a]	1.9	0.5	0.3
	285	2.3[a]	1.9	0.6	0.3
Diameter = 20.3 cm	0	0.9	0.8	10.7	9.8
Length = 1,654 m	41[b]	1.2	1.0	2.2	1.7
Flow = 1,907 m³/d					

[a] Weighted averages.
[b] Maximum injection rate limited by system hydraulics.

Table 3-9. Performance of Direct Oxygen Injection into Force Main at Sacramento, CA (7)

Oxygen Dosage	Total H_2S Concentration	
	Wet Well	Downstream Sample Tap
mg/l	mg/l	
0	0.16	2.14
18.8	0.12	0.22
37.5	0.07	0.13
45.6	0.09	0

Force main characteristics:
Diameter = 53 cm and 66 cm
Flow = 11,920 m³/d
Distance to sampling tap = 2,930 m

A U-tube oxygen injection system was evaluated at Port Arthur, Texas. Oxygen transfer efficiencies were lower than expected. This was attributed to errors in estimating pump capacity, resulting in a detention time in the U-tube of only 17 seconds compared to the design detention time of approximately 30 seconds (2).

Results of limited testing conducted on an oxygenated U-tube in Sacramento, California are summarized in Table 3-10 (7). Use of the oxygen U-tube resulted in a 60- to 70-percent reduction in total sulfide concentration at a point 1,000 m (3,300 ft) downstream (7).

c. Multiple U-Tube Dissolution

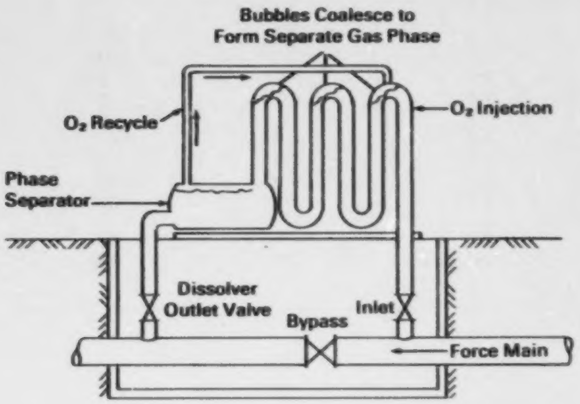
A proprietary multiple U-tube oxygen dissolution system has been successfully applied in Australia for H_2S control in force mains and gravity sewers, and for in-sewer wastewater treatment (19). A three-stage U-tube dissolver is shown in Figure 3-15 (21). Wastewater flows through the U-tubes creating a series of waterfalls which generate oxygen bubbles. Bubbles tend to reform at the top of each column in gas pockets. Wastewater flowing through the pockets under turbulent conditions results in partial dissolution of the accumulated oxygen. Travel through successive gas pockets increases the amount of oxygen dissolved in the wastewater. The oxygen entrainment rate (oxygen flow divided by liquid flow) through a three-stage U-tube is approximately 10

Table 3-10. Performance of Oxygen U-Tube for Sulfide Control at Sacramento, CA (7)

Flow	O_2 Supplied		Total Sulfide			DO		
			In	Out	1,000 m Downstream	In	Out	1,000 m Downstream
m³/d	m³/d	mg/l	mg/l			mg/l		
8,300	0	0	0.61	0.70	0.67	0	0	0.24
10,300	130	16.8	0.55	0.43	0.28	0	9.1	1.0
9,000	240	35.5	0.53	0.45	0.21	0	12.4	4.0
10,640	355	44.3	0.66	0.59	0.23	0	>20	9.0

percent by volume. Because this exceeds the oxygen solubility, undissolved oxygen is collected in a phase separator and recycled back to the first stage of the dissolver.

Figure 3-15. **Multiple U-tube dissolution system (1).**



Limited performance data for a multiple U-tube oxygen injection system installed near the end of a 60-cm (24-in) force main in Australia are presented in Table 3-11 (19). Because of the limited detention time prior to discharge and the resulting inefficiency in dissolution, the pressurized multiple U-tube dissolver was recommended.

Table 3-11. **Performance of Multiple U-Tube Oxygen Injection (19)**

	Winter Trial	Summer Trial
Flow, m^3/d	18.2	15.9
Injected O_2, mg/l		
Day	34	54
Night	185	280
Sulfide removed, mg/l	20	40
Total sulfide removed, kg/d	364	637
Ratio of O_2 used to sulfide removed	2.3:1	1.8:1
DO at headworks, mg/l	10	10

d. Pressurized Sidestream Injection

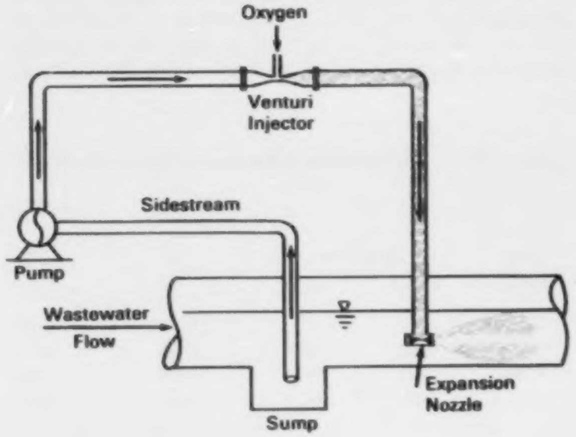
One technique for injecting oxygen involves saturating a portion of the flow under pressure with oxygen, then introducing the oxygen-enriched sidestream back into the main flow through a submerged nozzle or jet. Using a pressurized sidestream dissolution system ensures that little or no oxygen comes out of solution in the sidestream, and the high velocity of the oxygenated liquid discharging from the nozzle results in good mixing and dilution with the main stream. This keeps bubble size to a minimum, allowing good oxygen dissolution.

There are two main types of pressurized sidestream dissolvers: the "basic" system in which all the gas added is dissolved in the sidestream; and the proprietary "Vitox" system in which, after equilibrium is reached, only part of the gas is dissolved in the sidestream, the remainder being dissolved when the stream is returned via a submerged nozzle or jet to the main flow. Both types can operate over a wide pressure range, although the basic sidestream dissolution system requires a higher pressure for equivalent oxygen dissolution. The high velocity of the returning oxygenated liquid results in good mixing and dilution with the main stream in both systems.

The basic system normally operates at pressures up to 700 kPa (100 psig) at which 250 mg/l of oxygen can be dissolved in the sidestream. However, the turbulence and pressure drop at the return nozzle causes effervescence and gas loss. Operation at lower pressures reduces effervescence problems, but oxygen dissolution is reduced proportionately.

The sidestream from the proprietary Vitox process reportedly requires only 210 kPa (30 psig) to dissolve 220 mg/l of oxygen. Dissolution of the gas remaining undissolved in the sidestream is achieved by creating a shock wave in the nozzle which returns the sidestream to the main stream. The shock wave generates clouds of microbubbles which are rapidly dispersed in the main stream by the turbulence of the jet. This system has been effective on shallow trunk lines and channel inlets. It requires less energy than the basic sidestream dissolution system to dissolve an equivalent amount of oxygen (22). A typical sidestream flow diagram is shown in Figure 3-16.

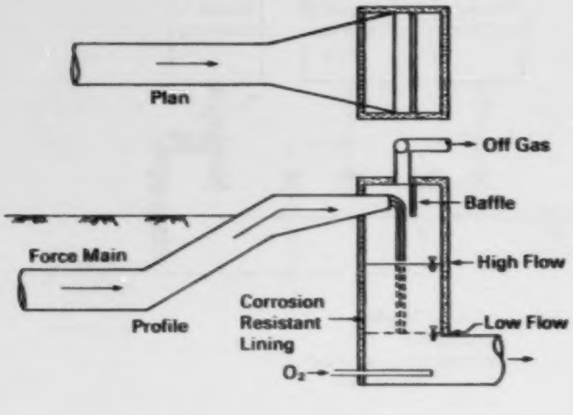
Figure 3-16. **Typical sidestream dissolution system (22).**



e. Hydraulic Fall Injection

Oxygen can be injected efficiently into a hydraulic fall due to the intense mixing and turbulence resulting from such structures. This can be an effective technique for oxygenation of force main discharges prior to entrance into gravity sewers. A diagram of oxygen injection at a hydraulic fall is shown in Figure 3-17 (7).

Figure 3-17. System for oxygen injection into a hydraulic fall (7).


Plan
Off Gas
Baffle
Force Main
High Flow
Profile
Corrosion Resistant Lining
Low Flow
O_2

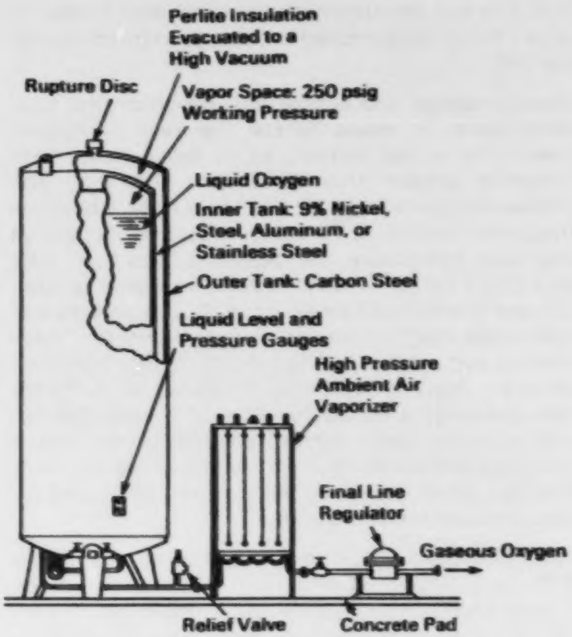
The turbulence caused by such a fall promotes oxygen transfer, but at the same time releases any H_2S gas to the wet-well atmosphere, thus requiring consideration of use of corrosion-resistant materials and collection and treatment of exhaust air. In one hydraulic fall oxygen injection system installation in Sacramento, California, H_2S and methane concentrations in an enclosed hydraulic fall reached levels considered to be hazardous to human health and constituted a potential explosion hazard. In an attempt to mitigate these problems, chlorine was added upstream of the fall to reduce sulfide levels. However, the oxygen injection system had not yet been commissioned as of January, 1984.

3.2.3.2 Oxygen Injection Equipment

Applications requiring less than approximately 900 kg (2,000 lb)/d of oxygen typically use purchased liquid oxygen. Where oxygen requirements exceed 900 kg (2,000 lb)/d, generation on-site using a Pressure Swing Absorption system may be more economical.

Equipment used for oxygen injection systems employing liquid oxygen includes a liquid oxygen storage vessel, a vaporizer, a pressure regulator, oxygen feed and injection piping, and a control system. For other than direct oxygen injection into force mains, separate dissolution equipment, such as U-tubes or sidestream pressurization systems, may be required. A typical layout of an oxygen injection station is shown in Figure 3-18 (23).

Figure 3-18. Typical liquid oxygen station (23).


Perlite Insulation Evacuated to a High Vacuum
Rupture Disc
Vapor Space: 250 psig Working Pressure
Liquid Oxygen
Inner Tank: 9% Nickel, Steel, Aluminum, or Stainless Steel
Outer Tank: Carbon Steel
Liquid Level and Pressure Gauges
High Pressure Ambient Air Vaporizer
Final Line Regulator
Gaseous Oxygen
Relief Valve
Concrete Pad

A liquid oxygen storage tank, which can be purchased or leased from the supplier, consists of an inner and outer vessel separated by an insulated annular space evacuated to 10 to 100 mm Hg. The inner tank is constructed of aluminum, stainless steel or high-nickel steel alloys. The outer vessel is carbon steel. Most tanks are designed to regulate pressure automatically. Excess gas pressure is relieved by release of vaporized oxygen into the discharge line, while low pressures are increased by vaporization of a portion of the liquid oxygen and recycle to the head space of the vessel.

The vaporizer is typically constructed of 304 stainless steel or aluminum with extruded fins, allowing vaporization of the liquid oxygen through exchange of heat from the ambient air. In some high-use applications, a low-temperature shut-off device is installed downstream of the vaporizer to prevent discharge of liquid oxygen and to provide protection of downstream equipment.

Oxygen piping is typically copper or stainless steel, although copper is more commonly used due to lower cost and ease of installation. These materials do not embrittle at low temperatures.

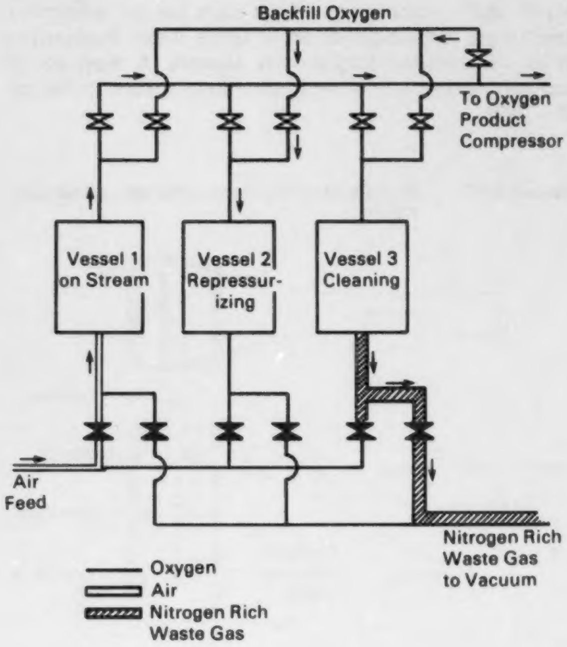
Selection of a control system is based on the type and sophistication of control desired. To prevent injection of excess oxygen, oxygen injection can be interlocked with pump starter circuits. A time delay can be designed to delay injection once pumping begins, and a timer used to limit the amount of oxygen injected during a pump cycle. Alternatively, rate of oxygen injection can be automatically controlled based on actual flows encountered in the system (proportional control).

Injector design and bubble size are important considerations. In cases where the rate of oxygen dissolution is not critical, as in force mains with velocities greater than 0.67 m/s (2.0 ft/s) and pressures greater than 15 m (49 ft) of head, a nozzle is frequently sufficient to achieve desired dispersion and local turbulence. For velocities less than 0.67 m/s (2.0 ft/s) or for lower pressures, a porous plate diffuser will achieve smaller bubble size and greater interfacial oxygen transfer surface. However, these devices are susceptible to biofouling and provision must be made for cleaning. Presence of sufficient mixing energy is critical for efficient oxygen dissolution. Unfortunately, current knowledge on oxygen injection into wastewater force mains does not allow precise calculation of the mixing energy required for complete oxygen dissolution.

For applications requiring greater than approximately 900 kg (2,000 lb)/d of oxygen, generation on-site using a Pressure Swing Adsorption (PSA) system may be more economical than purchasing liquid oxygen. A flow diagram for a PSA system is shown in Figure 3-19 (24). Pressure Swing Adsorption employs a cycle which operates between two pressures, adsorbing and separating the gas stream at the higher pressure and desorbing and exhausting waste products at the lower pressure. Separation of nitrogen and other contaminants occurs in a vessel by selective adsorption using a zeolite molecular sieve. This is a batch process, but cycling between multiple adsorption vessels allows reasonably constant oxygen flow. A product storage vessel is normally incorporated for further attenuation of the cyclic oxygen generation.

A timer and pressure switches control valves on a manifold, which directs pressurized feed air from a compressor to the adsorption vessels. The only moving parts are the valves and air compressor, which require periodic maintenance. The compressor operates either fully loaded (100-percent flow) or unloaded (no flow) and repressurizes a bed in approximately 1 minute.

Figure 3-19. Pressure swing absorption—basic flow diagram.



Equipment requirements for a PSA oxygen-generating station consist of a single-stage reciprocating, two-stage centrifugal, or screw compressor. Additional equipment includes inlet air filter-silencer, piping and valves, adsorbent vessel designed to meet ASME Code requirements using SA-516 Firebox, Grade 65 steel or equivalent, an integrated valve assembly interfaced with feed air, adsorbent vessels, an oxygen injection system, and appropriate instrumentation and controls. Some processes also use a vacuum pump. An evaporative cooling tower is provided to lower the temperature of the recirculated cooling water used to cool the air discharged from the compressor. In addition, liquid oxygen storage is normally provided as a back-up oxygen supply.

For oxygen injection into a gravity sewer or at transitions from pressure to gravity conditions, a separate oxygen dissolution system may be required to improve dissolution efficiency and prevent degassing. Systems previously described include U-tubes (single and multiple), and pressurized side-stream (high or low pressure) oxygen dissolution systems. Equipment requirements will vary depending on the specific application and the dissolution system selected.

3.2.3.3 Design of Oxygen Injection Systems

The design of an oxygen injection system is similar to the design of an air injection system in that the first step is to estimate oxygen requirements. Collection of sufficient data to estimate oxygen reaction rates is necessary. One design approach follows the stepwise procedures outlined in Section 3.2.2.3 for air injection systems, where the oxygen requirements are based on oxygen uptake in the wastewater stream and within the slime layer. This procedure can be applied to the design of oxygen injection facilities for both force main and gravity sewer applications.

An alternative design approach for direct oxygen injection into force mains is to use the following equation, which yields the initial DO level required to maintain aerobic conditions (25).

$$C_o = R_r + \frac{280}{d}(0.785\ d^2) \times \frac{L}{F} \times 10^{-4} \qquad (3\text{-}3)$$

where,

C_o = required oxygen concentration, mg/l

R_r = oxygen reaction rate, mg/l

280 = factor corresponding to an assumed oxygen reacting rate on the slime layer equal to 0.7 g/m^2/hr

d = pipe diameter, cm

L = pipe length, m

F = discharge, m^3/hr

10^{-4} = conversion factor, cm^2 to m^2

The total daily oxygen requirement can then be determined as:

$$\text{kg } O_2/\text{day} = \frac{F \times C_o \times 24}{1{,}000} \qquad (3\text{-}4)$$

Many of the design considerations that apply to air injection also apply to pure oxygen injection. These include location of oxygen injector, control of gas pocket formation, and options for control systems.

Gas pockets are likely to form in any two-phase system, and the potential always exists for gas locking of pumps. However, problems associated with gas pocket formation are less likely to occur with oxygen injection, since the volume of oxygen gas necessary to add 1 kg of oxygen is approximately one-fifth the volume of air required to add an equivalent amount of oxygen. However the same precautions should be observed as those outlined in Section 3.2.2.3 in order to prevent such occurrences.

a. Single U-Tube

Design criteria for single U-tube installations have been described for air injection applications in Section 3.2.2. In general, design of an oxygen U-tube is approached in the same manner. Oxygen is injected at the top of the downleg to maximize contact time with the wastewater to allow dissolution to occur. Since the volume of oxygen is approximately one-fifth the volume of air to achieve the same mass of oxygen transferred (assuming complete dissolution), head losses with oxygen U-tubes are considerably less than with compressed air or aspirated air U-tubes. Figure 3-20 shows the relationship between DO increase across a U-tube and head loss for both air and oxygen U-tubes (7).

Because oxygen must be purchased unless generated on-site, it is desirable to make efficient use of the oxygen. Oxygen U-tubes should, therefore, be designed for greater depths to promote maximum dissolution. Volumetric gas:water ratios for oxygenated U-tubes are generally about 0.02, corresponding to an injection rate of 20 to 30 mg/l.

Figure 3-21 shows an expected performance curve for an oxygenated U-tube designed to handle a flow of 6,540 m^3/d (1.7 mgd). This shows the relationship between oxygen supplied and oxygen transferred for U-tubes of various depths, as well as expected head losses vs. oxygen supplied for various U-tube depths. The family of curves at the top of Figure 3-21 shows the ratio of oxygen transfer with pure oxygen to oxygen transfer with air, the ratio ranging from 3.5 to 4.5.

b. Multiple U-Tube

The proprietary multiple U-tube system is different from the conventional U-tube and can be considered as much a mixing device as a dissolver. As opposed to conventional U-tubes, the length of the vertical legs in the multiple U-tube is only 1.8 to 2.4 m (6 to 8 ft). Velocities through the uniformly sized pipe are normally 0.5 to 1.2 m (1.5 to 4.0 ft)/s, although they may be as high as 2.4 m (8.0 ft)/s. Figure 3-15 shows the application of the system for force main oxygenation, where the existing pressure in the force main

Figure 3-20. Dissolved oxygen increase in the wastewater across U-tube vs. head loss comparing oxygen and air injection (7).

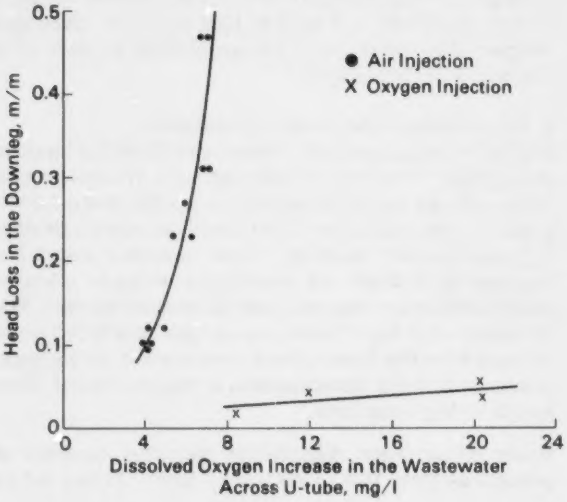
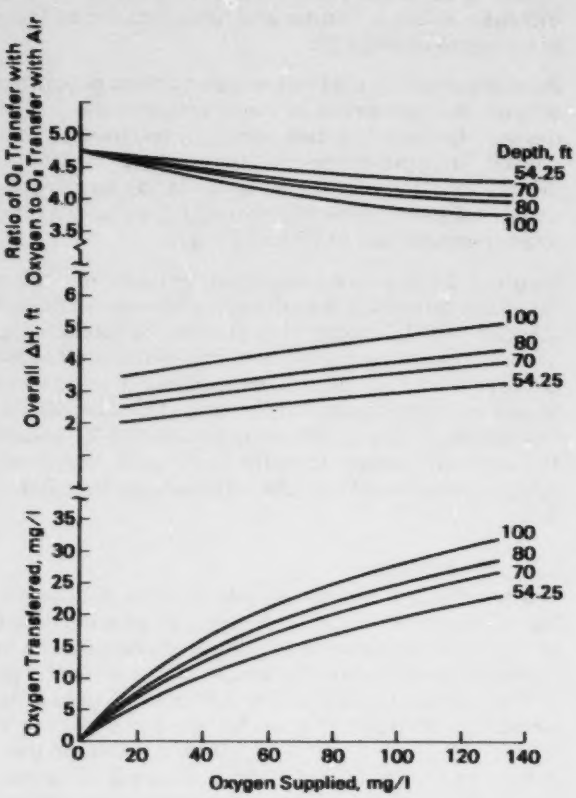


Figure 3-21. Typical design curve for oxygen U-tube (7).



drives the wastewater through the dissolution system. However, the system can also be designed as a pressurized sidestream dissolution system for application to gravity sewers or at transitions from pressure to gravity conditions by employing high pressure pumps. Design head loss across the multiple U-tube dissolver is 2 to 3 m (6.5 to 10 ft). Expected oxygen dissolution will be equivalent to that of a water fall of this height.

c. Pressurized Sidestream Dissolution

Sidestream dissolution systems can be of the basic or proprietary Vitox type. Basic systems are only applicable to force mains in which the profile and ambient pressure are sufficient to prevent excessive oxygen degassing after injecton. Vitox systems avoid the degassing problem at equivalent oxygen addition rates and are also appropriate for gravity sewers. The injection of a high pressure oxygen enriched sidestream from the basic sidestream system into wastewater at or near atmospheric pressure would likely result in high gas loss.

Basic sidestream dissolution systems operate at pressures from 105 to 700 kPa (15 to 100 psi), while Vitox systems usually operate at pressures of 210 kPa (30 psi) (22). Sidestream flow rates will depend on oxygen requirements of the wastewater and incoming sulfide concentration.

A Vitox sidestream system approaches 40- to 60-percent oxygen transfer efficiency at an injection rate of 10 to 30 mg/l in a shallow channel (22).

d. Hydraulic Fall Injection

A hydraulic fall is an excellent location for oxygen injection due to the intense mixing and turbulence resulting from dissipation of the kinetic energy of the wastewater. Often oxygen can be injected into a hydraulic fall at the transition from a force main to a gravity sewer.

Dissolution efficiencies for hydraulic falls can be estimated using the following relationship:

$$\ln \frac{D_1}{D_2} = K_H (H_1 - H_2) \qquad (2\text{-}15)$$

where,

D_1 and D_2 = oxygen deficits upstream and downstream, respectively, from the drop, mg/l

K_H = waterfall reaeration coefficient, m^{-1} (approx. 0.41)

H_1 and H_2 = elevations of the hydraulic energy lines upstream and downstream, respectively, from the drop, m

This equation was used to develop Table 2-6, which predicts the percent of the oxygen deficit that would be satisfied for falls of various elevations. This table can be used to generate a first estimate of the efficiency of dissolution if pure oxygen is injected. However, an assumption must be made regarding the fraction of oxygen that is wasted in such an injection scheme. For example, for a drop of 3 m (10 ft), a 71-percent oxygen dissolution efficiency might be expected (Table 2-6). If 30 mg/l of oxygen is applied, of which 20 percent is assumed to be lost or wasted, the resulting DO concentration can be estimated to be 30 x 0.71 x (1 -0.20) = 17 mg/l. In general, whenever DO levels are raised above the equilibrium value with air (8 to 10 mg/l), then oxygen gas is lost in removing some of the dissolved nitrogen which is in excess.

An important design consideration in any hydraulic fall is the release of H_2S resulting from the intense turbulence. If sulfides are present in the incoming flow, significant odor and corrosion may result. Thus, it may be necessary to provide for corrosion protection of the structure and for removal and scrubbing of the H_2S-laden atmosphere. Consideration should be given to the accumulation of potentially hazardous or explosive gases, and to whether the enclosed structure can be adequately ventilated to prevent such problems.

3.2.3.4 Cost of Oxygen Injection Systems

Typical costs of systems for direct oxygen injection into a force main are shown in Table 3-12. These costs are budget level estimates (+30 percent, –15 percent). Additional costs would be incurred if separate oxygen dissolution systems were employed. For some of the dissolution alternatives (e.g., U-tubes), costs are largely dependent on site conditions and resulting site-specific design criteria. Other dissolution techniques, such as multiple U-tube systems, have not been used in the United States, and as a result, few definitive cost data are available. Thus, cost estimates are not given for these alternatives.

It should be noted that the capital cost estimates shown in Table 3-12 assume purchase of equipment by the user. In many cases, oxygen suppliers will lease the necessary storage tanks, vaporizers, etc. at rates that will result in equipment lease being economically advantageous over direct purchase. In estimating costs for an oxygen injection system, oxygen suppliers should be contacted to determine costs for leasing of equipment.

3.3 Chemical Addition

Numerous chemicals have been employed for control of sulfides in wastewater collection systems. Chemical addition can control sulfides by: 1) chemical oxidation (Cl_2, H_2O_2); 2) sulfate reduction inhibition by providing an additional oxygen source (NO_3^-); 3) precipitation (metal salts); or 4) pH control (strong alkalies).

3.3.1 Chlorine

Chlorine may be added to wastewater either as hypochlorite or chlorine gas. Hypochlorite may be used where applications are occasional or dosages are small, but economies of scale dictate use of chlorine gas where larger quantities, i.e., >2.3 kg (5 lb)/d, are necessary.

Chlorine combines with water to form hypochlorous and hydrochloric acids, as follows (26):

$$Cl_2 + H_2O \longleftrightarrow HOCl + H^+ + Cl^- \qquad (3\text{-}5)$$

The dissociation of HOCl is shown as:

$$HOCl \longleftrightarrow H^+ + OCl^- \qquad (3\text{-}6)$$

The equilibrium constant for this reaction at 20°C is pK = 7.57.

When calcium hypochlorite is added to water, it ionizes to yield hypochlorite ion:

$$Ca(OCl)_2 \longrightarrow Ca^{++} + 2OCl^- \qquad (3\text{-}7)$$

Table 3-12. Typical Costs for Direct Oxygen Injection into Force Main for H_2S Control (1984 $)

Condition	Capital Cost[a]	Annual Chemical (O_2) Costs
	$	$/yr
Flow = 3,785 m^3/d Pipe: diam = 36 cm length = 1,600 m O_2 req'd = 83 kg/d	20,000	9,000[b]
Flow = 37,850 m^3/d Pipe: diam = 76 cm length = 1,600 m O_2 req'd = 310 kg/d	50,000	20,000[c]

[a] Includes concrete pad, liquid O_2 storage vessel, vaporizer, piping, start-stop controls, and installation.

[b] Based on O_2 cost of $0.31/kg.

[c] Based on O_2 cost of $0.18/kg.

The OCl^- ion establishes an equilibrium with hydrogen ions in accordance with Equation 3-6. The same equilibria are established whether chlorine is added as a gas or as hypochlorite, although pH may be affected. Chlorine gas tends to reduce the pH, while hypochlorite tends to increase the pH.

If excess chlorine is added to a wastewater containing sulfide, sulfide is oxidized to sulfate according to the following reaction (14):

$$HS^- + 4Cl_2 + 4H_2O \longrightarrow SO_4^= + 9H^+ + 8Cl^- \qquad (3\text{-}8)$$

For this reaction, 8.87 parts by weight of chlorine are required to oxidize each part of sulfide. If chlorine is added slowly to a pure sulfide solution under vigorous mixing conditions, sulfide is oxidized to sulfur as described by the following reaction (14):

$$HS^- + Cl_2 \longrightarrow S + H^+ 2Cl^- \qquad (3\text{-}9)$$

Chlorine consumption in this reaction is 2.22 parts chlorine per part of sulfide. In laboratory tests conducted by the Los Angeles County Sanitation District, the chlorine requirement for elimination of sulfides was three to nine times the sulfide concentration. Actual observations in full-scale studies showed somewhat higher dosages, possibly due to inefficient mixing (27).

Table 3-13 shows data from a field testing program in Sacramento, California, where chlorine was added to a pump station located at the beginning of a 1,520-m (5,000-ft) long, 61-cm (24-in) diameter force main (7). Average dry weather flow at the pump station was 7,570 m^3/d (2.0 mgd). Data were collected at chlorine application rates of 500 kg (1,100 lb)/d, 270 kg (600 lb)/d, and 135 kg (300 lb)/d. Based on actual flow rates, this was approximately equivalent to dosages of 46 mg/l, 28 mg/l, and 14 mg/l, respectively. As shown by Table 3-13, all three application rates were effective in reducing total sulfide concentrations to zero.

Table 3-13. Performance of Chlorination for Sulfide Control at Sacramento, CA (7)

Distance from Injection Point	Approximate Flow Time from Cl_2 Injection Point	Average Cl_2 Residual with Cl_2 Dosage of:[a] 46	28	14	Average H_2S Concentration: Without Chlorination	With Chlorination[b]
m	hours		mg/l		mg/l	
1,700	1.4	38.5	17.5	8.0	0.50	0
2,700	1.8	42.0	17.4	4.3	0.60	0
3,300	1.9	3.6	.05	0	0.32	0
4,100	2.1	2.2	0	0	0.24	0
6,000	2.8	0.3	0	0	0.19	0

[a] Values for Cl_2 residual are an average of two to five measurements.
[b] The three feed rates were studied over a total of five days. At each downstream sampling site there were 15 to 20 sulfide determinations over this time and no sulfide was found.

A study was conducted in Tampa, Florida to evaluate the effectiveness of chlorination for sulfide control in a force main (28). Prior to chlorine addition, total sulfide levels in the raw wastewater were as high as 14 mg/l. Detention times in the section of force main studied were 10 to 17 hours. Average year-round temperature of the wastewater was 28°C. Flows in the force main typically ranged from 18,900 to 22,700 m^3/d (5 to 6 mgd). Table 3-14 summarizes the results of this study. It can be seen that at average chlorine dosages of 35 to 40 mg/l, sulfide concentrations were reduced to zero at a point 6.5 km (4.0 mi) downstream of the injection point, and were reduced by about 60 percent at a point 13.7 km (8.5 mi) downstream of the injection point.

3.3.1.1 Equipment Required

Table 3-15 summarizes the characteristics of chlorine available as liquid or gaseous chlorine and as hypochlorite (29).

Hypochlorite feed systems are generally used when the requirement for chlorine is less than 2.3 kg (5.0 lb)/d. If the dosage were 10 mg/l, this would correspond to chlorinating a flow of about 230 m^3 (60,000 gal)/d. Equipment required for hypochlorite addition includes: 1) a hypochlorite storage vessel; 2) a metering pump; and 3) a diffuser or injector.

Table 3-14. Effects of Chlorination on H_2S Concentrations in Force Main at Tampa, FL (28)

Chlorine Feed Rate	Average Cl_2 Dose	H_2S Concentrations: Injection Point	4.6 m	6.5 km	13.7 km
kg/d	mg/l		mg/l		
0	0	0.4	1.8	7.0	9.0
455 (constant)	21.6	0.7	0	4.6	6.8
455 (12 a.m.-7 a.m.) 910 (7 a.m.-12 a.m.)	35.0	0.8	0	0	3.7
682 (12 a.m.-7 a.m.) 910 (7 a.m.-12 a.m.)	38.0	0.8	0	0	3.7
910 (constant)	40.0	0.6	0	0	3.5

Most applications of chlorination for sulfide control employ gaseous chlorine systems. A typical chlorination system, shown in Figure 3-22, consists of a chlorine storage facility chlorine scale, a water supply, a chlorinator, and a diffuser for injecting the chlorine solution into the stream (29). Figure 3-23 presents a typical design of chlorine diffusers for gravity sewers and force mains (30). Booster pumps are generally required when chlorine is injected into a force main. For installations requiring greater than 230 kg (500 lb)/d, a separate evaporator is used.

Several control options are available for chlorine feed systems. The simplest and most inexpensive is a manually controlled system in which chlorine dosages are adjusted manually and the feed rate is constant. Control systems that are particularly useful for installations lacking flow measuring devices are: 1) varying dosage by a preset cam controller based on a diurnal chlorine requirement; and 2) setting chlorinators to dose in increments based on the number of pumps operating (30). Somewhat more sophisticated control systems include proportional control, Cl_2 residual control, and compound loop control. Proportional control adjusts chlorine feed based on an electrical signal from a flowmeter. Chlorine residual control adjusts the chlorine feed as necessary to meet a desired chlorine residual immediately downstream of the injection point. Finally, compound loop control adjusts chlorine feed based on both flow rate and residual chlorine concentration.

3.3.1.2 Design

Design of a chlorination system for sulfide control is dependent upon objectives of sulfide control (target levels of sulfide at a given point downstream), initial sulfide concentration, characteristics of the waste-

Table 3-15. Commercially Available Forms of Chlorine for Wastewater Applications (29)

Chemical	Shipping Containers	Handling Materials	Available Form	Commercial Strength	Characteristics
				%	
Liquid, gaseous chlorine (Cl_2)	45-, 70-, and 900-kg containers; tank cars	Steel	Liquid, gaseous	100	Liquid, vapor at atmospheric conditions
Sodium hypochlorite (NaOCl)	20- and 50-l carboys; 4,900- to 7,500-l tank trucks	Ceramics, glass, plastics, rubber	Light yellow liquid	12 to 15 (avail. chlorine)	Deteriorates with time
Calcium hypochlorite ($CaOCl_2$)	Small cans (7 kg); 45-, 135-, and 365-kg drums; 190-kg bbl.	Glass, rubber, stoneware, wood	White granular powder	70 (avail. chlorine)	1 to 3 percent available chlorine solution used

Figure 3-22. Typical chlorinator installation (29).

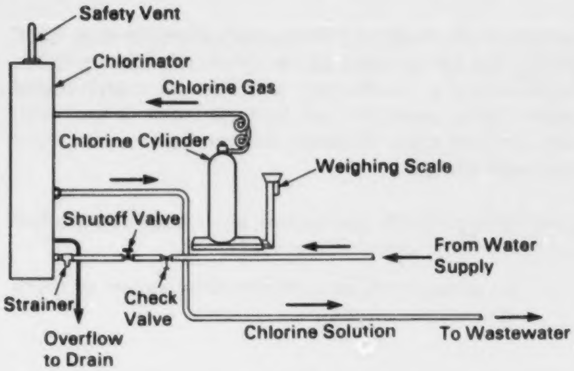
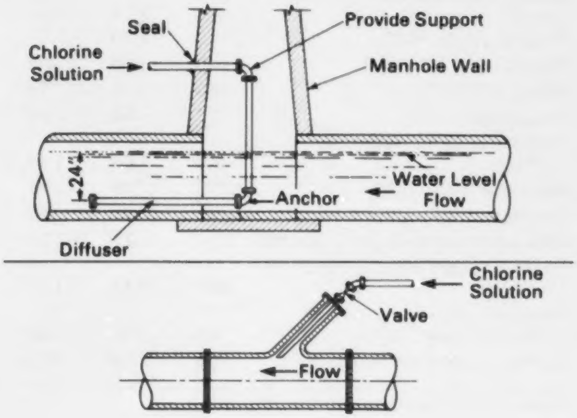

Safety Vent
Chlorinator
Chlorine Gas
Chlorine Cylinder
Weighing Scale
Shutoff Valve
From Water Supply
Strainer
Check Valve
Overflow to Drain
Chlorine Solution
To Wastewater

Figure 3-23. Typical chlorine diffusers for gravity sewers and force mains.


Seal
Provide Support
Chlorine Solution
Manhole Wall
24"
Water Level
Anchor
Flow
Diffuser
Chlorine Solution
Valve
Flow

water (presence of other chlorine-demanding constituents), and the degree of chlorine feed control desired.

At a minimum, laboratory studies should be conducted to estimate the chlorine dosage required to produce the desired sulfide reduction. Field application rates have generally been 10 to 15 kg Cl_2/kg H_2S oxidized. Since laboratory data are not easily applied to full-scale design due to variations in flow rate, sulfide concentration, wastewater characteristics, and mixing energy at the point of injection, flexibility in feed rate is necessary. Ideally, full-scale studies should be performed over a period of several weeks to determine the effectiveness of chlorine in achieving the specific objectives in sulfide control for the locations of interest.

Provision of intense mixing at the point of chlorine injection is critical for efficient chlorination. It is likely that failure to provide adequate mixing in full-scale applications accounts for the discrepancies between laboratory and full-scale chlorine requirements. One method used to achieve good mixing is to inject the chlorine solution into a hydraulic fall of 0.3 m (1 ft) or greater, or into a hydraulic jump. Poor mixing and sluggish, shallow flows are likely to cause fuming from localized chlorine overdose.

Chlorine is a hazardous material. Design of a chlorination station must recognize the need to make appropriate provisions for the safe handling and storage of chlorine (29). Another concern which has been raised is the potential formation of toxic or carcinogenic chlorinated hydrocarbons during wastewater chlorination. These factors should be considered in the evaluation of sulfide control alternatives.

3.3.1.3 Costs

Typical costs for a chlorine injection system are presented in Table 3-16. These costs are budget level

Table 3-16. **Typical Costs for Chlorine Injection for H_2S Control (1984 $)**

Condition	Capital Cost[a]	Annual Chemical (Cl_2 Costs[b])
	$	$/yr
Flow = 3,785 m^3/d $[H_2S]$ = 5 mg/l Cl_2 dose = 30 mg/l	18,000	14,000
Flow = 37,850 m^3/d $[H_2S]$ = 5 mg/l Cl_2 dose = 30 mg/l	40,000	137,000

[a] Includes concrete pad, chlorinator, booster pump, scale, piping, safety equipment, start-stop controls, building and vaporizer (larger system only), and installation.

[b] Based on typical cost of chlorine in 900 kg (1 ton) cylinders of $0.33/kg.

cost estimates (+30 percent;-15 percent). A simple control scheme, consisting of chlorinator interlock with pump starter circuits at a pump station, has been assumed for simplicity. More sophisticated control schemes, such as flow proportional or chlorine residual control, were not considered appropriate for application to a remote pumping station in a collection system. Other simple control schemes, such as timer control or preset cam control, would be equally appropriate for such applications.

3.3.2 Hydrogen Peroxide

Hydrogen peroxide chemically oxidizes H_2S according to the following reactions:

$$pH < 8.5: H_2O_2 + H_2S \longrightarrow S + 2H_2O \quad (3\text{-}10)$$

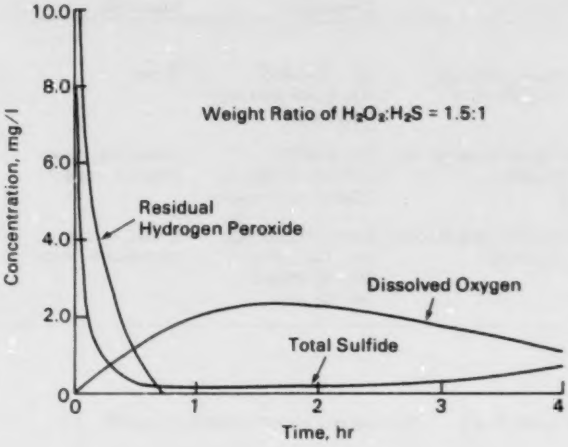
$$pH > 8.5: 4H_2O_2 + S^{=} \longrightarrow SO_4^{=} + 2H_2O \quad (3\text{-}11)$$

At pH < 8.5, the stoichiometric H_2O_2 requirement is 1 g H_2O_2/g H_2S. In practice, a somewhat greater weight ratio may be employed, depending on whether the application is only for oxidizing existing sulfides or for preventing additional sulfide formation. For the latter, dosage rates will vary with BOD and temperature of the wastewater, and with hydraulic characteristics of the sewer.

The reaction of H_2O_2 with H_2S is rapid. Figure 3-24 shows a typical reaction profile for H_2O_2 in wastewater. Generally, 90 percent of the peroxide is reacted within 10 to 15 minutes, with the reaction completed in 20 to 30 additional minutes (31).

Hydrogen peroxide is commercially available as solutions of 35-, 50-, and 70-percent H_2O_2 by weight. The physical properties of hydrogen peroxide are summarized in Table 3-17. Bulk shipments of hydrogen peroxide are as 50- or 70-percent H_2O_2 by weight. For storage, the 70-percent solution is diluted to 50 percent, since at the higher concentrations, hydrogen peroxide becomes more hazardous to handle. For safety reasons, 50-percent solutions are typically purchased for sulfide control applications in populated areas. For small package installations, 230-kg (500-lb) drums are purchased, while larger installations rely on shipments in bulk by tank truck or tank car. Storage tank capacities vary depending on anticipated peroxide usage.

Figure 3-24. **Typical reaction profile of hydrogen peroxide in wastewater (31).**



Hydrogen peroxide has certain advantages over other sulfide control alternatives (31):

1. It can be used for either gravity sewer or force main applications.

Table 3-17. **Physical Properties of Hydrogen Peroxide**

Parameter	H_2O_2 Concentration by Weight, Percent		
	35	50	70
Volume strength, 0°C and 1 atm	130	197	300
Active oxygen content, percent by weight	16.5	23.5	32.9
Specific gravity, 20°C	1.13	1.20	1.29
Density, kg/m^3 @ 20°C	1,126	1,198	1,294
Boiling point, °C	108	114	126
Freezing point, °C	-33	-52	-40
Viscosity at 25°C, centipoise (mPa/s)	1.0	1.06	1.12
Refractive index, 25°C	1.355	1.366	1.381
Dielectric constant, 20°C	83	83	82
Total vapor pressure, 30°C (mm Hg)	23.3	18.3	10.1
Partial pressure of H_2O_2, 30°C (mm Hg)	0.28	0.56	1.17
Heat of dilution, cal/g mole of H_2O_2, 25°C and 1 atm	-84	-178	-381
Surface tension, dynes/cm, 20°C	74.6	75.6	77.3

Appearance: colorless, odorless liquid

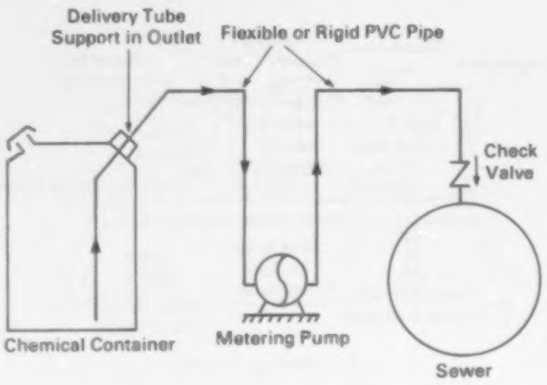
2. The chemical feed system is relatively simple and inexpensive.
3. The reactions with sulfide or other wastewater constituents produce harmless by-products.
4. The decomposition of excess H_2O_2 results in addition of DO to the stream.
5. With proper dosage, H_2S generation will be suppressed for 3 to 4 hours after H_2O_2 addition.

Use of hydrogen peroxide for sulfide control has been successfully demonstrated at numerous locations in the United States, the United Kingdom, Australia, and elsewhere (32-41). Table 3-18 summarizes data from five locations in the United States where hydrogen peroxide has been used for sulfide control.

3.3.2.1 Equipment

Equipment required for hydrogen peroxide addition is relatively simple, consisting of a storage vessel, metering pumps, appropriate valving and transfer piping, and injection nozzle. For small or intermittent applications, a package dosing system as shown in Figure 3-25 can be used. In this system, hydrogen peroxide is withdrawn directly from containers delivered by the supplier. Figure 3-26 illustrates the design of a bulk H_2O_2 feed installation for a force main. Such larger installations generally employ pneumatically or electrically activated ball valves operated in unison with the metering pump. The valve is open only when the metering pump is operating, preventing backflow of wastewater into the H_2O_2 feed piping. A pressure relief valve is provided in the event of failure of the ball valve in the closed position. In addition, a check valve must also be used for additional assurance against backflow of wastewater and contamination of the H_2O_2 feed piping.

Figure 3-25. Typical package H_2O_2 dosing installation (32).



Because hydrogen peroxide is a strong oxidant, specification of materials for storage and metering of the chemical is important. Bulk storage tanks are typically of high purity aluminum alloy construction. Smaller drums, 230-kg (500-lb), are polyethylene-lined steel, polyethylene with steel overpack, or self-supporting plastic barrels. Aluminum, stainless steel, or PVC is used in piping. Diaphragm-type metering pumps are generally used, and must be constructed of materials resistant to H_2O_2 exposure, such as aluminum alloys and Teflon. Hydrogen peroxide is very sensitive to contamination by many materials, which may cause rapid degradation of the H_2O_2. A field cleansing procedure known as passivation is essential for new equipment installations, or where subsequent maintenance work has potentially contaminated the storage or metering systems. Passivation consists of successive washings with a detergent, a solvent, water, nitric acid, and water (38).

3.3.2.2 Design

Ratios (by mass) of peroxide addition to incoming sulfides have typically ranged from 0.9 to over 3.0,

Table 3-18. Performance of H_2O_2 for Sulfite Control in Wastewater Collection Systems[a]

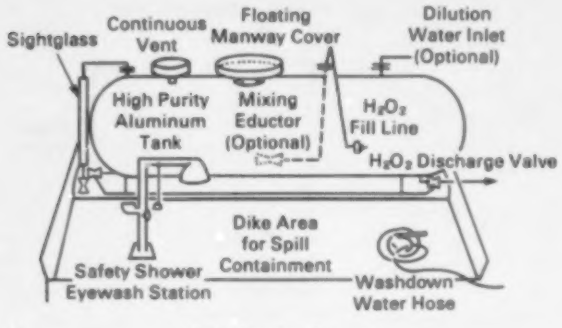
Plant Location	Average Daily Flow	Untreated Influent H_2S	Average H_2O_2 Dose		Time - H_2O_2 Dosed Upstream From Headworks	Residual H_2S	
						Headworks	Primary Weirs
	m^3/d	mg/l	mg/l	kg/d		mg/l	
Fort Worth, TX	227,100	7.9	9.3	2,120	10-15 minutes	0.7	1.7
Orange County, CA	662,400	5.2	7.8	5,160	6-9 minutes	1.1	0.5
		5.2	5.5	3,650	6-9 minutes	1.6	1.1
Baltimore, MD	673,700	0.95	2.0	1,360	4 minutes	NT[b]	0.5
	673,700	1.47	3.0	2,020	4 minutes	NT[b]	0.7
Gainesville, FL	24,600	19.2	16	390	60 minutes	1.8	NT[b]
West Palm Beach, FL	45,400	11	15 total[c]	690	7 hours total[c]	1.0	NT[b]

[a] Data compiled from internal data and demonstration reports, Interox America, Houston, TX.

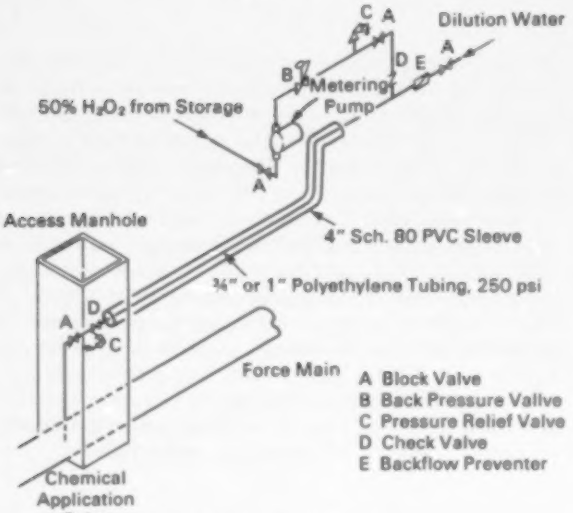
[b] NT: No testing was conducted at these points.

[c] 2 dosing sites: 1) 7 hours upstream from plant influent; 2) 3.5 hours from plant influent.

Figure 3-26. Typical bulk H_2O_2 storage and injection system.


Sightglass
Continuous Vent
Floating Manway Cover
Dilution Water Inlet (Optional)
High Purity Aluminum Tank
Mixing Eductor (Optional)
H₂O₂ Fill Line
H₂O₂ Discharge Valve
Dike Area for Spill Containment
Safety Shower Eyewash Station
Washdown Water Hose

(a) Bulk Storage System


C A
Dilution Water
B
D E
A
Metering Pump
50% H₂O₂ from Storage
A
Access Manhole
4" Sch. 80 PVC Sleeve
¾" or 1" Polyethylene Tubing, 250 psi
A
D
C
Force Main
A Block Valve
B Back Pressure Vallve
C Pressure Relief Valve
D Check Valve
E Backflow Preventer
Chemical Application Point

(b) Injection System

depending on the objectives for sulfide control (removal of existing sulfides or prevention of sulfide buildup), wastewater BOD, and pipe characteristics. Because of the variations in flow and incoming sulfide mass, metering pump capacities should be selected to provide adequate flexibility in feed rates. It is good practice to specify a pump having a maximum discharge of twice the anticipated feed rate, and to specify a pump adjustable over a range of 10 to 100 percent of its maximum capacity (37). Multiple pumps integrated into a manifold system can be operated on a simple timer circuit to increase feed rate during periods of high influent sulfides.

In order to properly specify the number and capacity of metering pumps, it is necessary to thoroughly characterize the diurnal flow rates and fluctuations in mass of sulfides in the system. It is prudent to optimize dosages in order to minimize chemical costs. Mass sulfide profiles similar to that shown in Figure 2-16 should be developed for each sewer reach where sulfides are to be controlled.

Laboratory tests should first be conducted to generate an initial estimate of the kinetic rate of the H_2O_2 reaction with sulfide. The reaction rate will be dependent on the wastewater characteristics and iron content. Testing is necessary to estimate the time required for reaction completion in order to select a suitable injection point.

In some cases, H_2O_2 suppliers will provide a range of professional services, including pilot studies, start-up services, and post construction monitoring. In addition, arrangements can be made for lease of equipment rather than direct purchase.

Adequate safety precautions must be observed in the handling of hydrogen peroxide. Protective clothing, including face shields, must be worn during bulk storage filling or during repair or maintenance work where contact or spillage might occur. Any spills should be immediately washed down with water to prevent spontaneous combustion of organic materials in the presence of H_2O_2. Emergency eye wash and shower facilities are necessary for maintenance personnel.

3.3.2.3 Costs

Typical costs of hydrogen peroxide injection systems are shown in Table 3-19. These are budget level estimates (+30 percent; -15 percent). It is assumed in this case that all the necessary equipment is purchased by the user. In practice, however, it is often economically advantageous to lease the equipment from a supplier. Some suppliers offer design, start-up and monitoring services for the municipality. Such organizations should be consulted in determining the cost effectiveness of hydrogen peroxide injection for sulfide control.

Table 3-19. **Typical Costs for Hydrogen Peroxide Injection for H_2S Control (1984 \$)**

Condition	Capital Cost[a]	Annual Chemical (H_2O_2) Costs
	\$	\$/yr
Flow = 3,785 m^3/d $[H_2S]$ = 5 mg/l H_2O_2 dose = 10 mg/l[b]	25,000	21,000[c]
Flow = 37,850 m^3/d $[H_2S]$ = 5 mg/l H_2O_2 dose = 10 mg/l	50,000	189,000[d]

[a] Includes concrete pad, H_2O_2 storage tank, metering pumps, piping, valves, timer controls, safety equipment, and installation.
[b] Dosage to control 5 mg/l H_2S at dosing station plus 2 mg/l H_2S formed downstream (2-hr detention).
[c] Based on H_2O_2 cost of \$1.50/kg as 100-percent H_2O_2.
[d] Based on H_2O_2 cost of \$1.36/kg as 100-percent H_2O_2.

3.3.3 Metal Salts

The salts of many metals will react with dissolved sulfide to form metallic sulfide precipitates, thus preventing H_2S release to the atmosphere. For effective removal of dissolved sulfides, the metallic sulfide formed must be highly insoluble.

Iron salts have been used for sulfide control at several locations in the United States (42). The ferrous ion reacts with sulfide as shown below:

$$Fe^{++} + HS^{-} \longrightarrow FeS + H^{+} \qquad (3\text{-}12)$$

Pomeroy found that the reaction of a mixture of iron salts with a molecular ratio of one part ferrous to two parts ferric was superior for sulfide control compared to the reaction of either one alone (27). The reaction of the mixed iron salts was hypothesized to occur as follows:

$$Fe^{++} + 2\ Fe^{+++} + 4HS^{-} \longrightarrow Fe_3S_4 + 4H^{+} \qquad (3\text{-}13)$$

Zinc salts have also been used for sulfide control. Zinc sulfide is much less soluble than iron sulfide, allowing theoretical reductions of sulfide concentrations to less than 0.1 mg/l. The effects of adding zinc solutions to reaches of force mains and gravity sewers have been studied. It was concluded that 10 to 15 parts of zinc would be required for every part of sulfide removed (3). The stoichiometric requirement is approximately 2:1. The city of Los Angeles, California, used zinc for sulfide control in a large trunk sewer. Solutions were prepared by dissolving scrap zinc in waste acid. This practice has since been discontinued (14).

Other metals, such as lead and copper, could also be used for sulfide control. However, the high costs and possible detrimental impact on downstream biological treatment processes generally eliminate their use from further consideration.

A mixture of ferric sulfate and nitric acid was successful in controlling sulfides in a force main in the United Kingdom, and was found to be more cost-effective than hydrogen peroxide or sodium nitrate addition. The mixture had a strength equivalent to 42,000 mg/l nitrate nitrogen and 125,000 mg/l ferric iron (43).

Of all the metal salts which have been or are being used for sulfide control, ferrous sulfate is the most common. One commercially available $FeSO_4$ solution is derived from the manufacture of titanium dioxide, which results in the production of ferrous sulfate crystal, and is sold for both sulfide control and phosphorus removal. Waste pickle liquor, resulting from the reaction of scrap iron with sulfuric acid, has also been used. However, such products may contain a high free acid content, which may result in detrimental impacts on wastewater pH and alkalinity.

Results from use of $FeSO_4$ for sulfide control are shown in Table 3-20.

Table 3-20. Performance of $FeSO_4$ Addition for Sulfide Control (42)

Location	Average Wastewater Flow	Average $FeSO_4$ Dosage[a]	Dissolved H_2S
	m^3/d	mg/l	mg/l
Clearwater, FL	18,900	0	6 to 8
		30	<1
Naples, FL	20,820	0	5 to 20
		25	1 to 2
Boyton Beach, FL	45,420	0	10
		25	0.5 to 2.0

[a] $FeSO_4$ solution used was a proprietary product derived from the manufacture of titanium dioxide, resulting in ferrous sulfate crystal.

3.3.3.1 Equipment

Equipment required for a $FeSO_4$ feed system is simple, and includes a storage tank, chemical metering pumps, piping and valves, control system, and injector. Materials suitable for a $FeSO_4$ storage tank include: 1) polyolefin; 2) concrete lined with polyurethane; 3) fiberglass; 4) steel lined with rubber; or 5) stainless steel.

Ferrous sulfate is only mildly corrosive, but safety precautions must be observed in its handling.

3.3.3.2 Design

The overall reaction of $FeSO_4$ with H_2S can be expressed as:

$$FeSO_4 + H_2S \longrightarrow FeS + H_2SO_4 \qquad (3\text{-}14)$$

Based on this reaction, removal of 1 g of H_2S would require approximately 4.5 g of $FeSO_4$ (1.6 g as Fe). Actual dosage requirements for a particular condition are determined by field application.

3.3.3.3 Costs

Typical costs for ferrous sulfate injection systems are shown in Table 3-21. Due to variability in site conditions, injection points, etc., capital costs are budget level estimates (+30 percent, -15 percent).

Table 3-21. Typical Costs for $FeSO_4$ Injection for H_2S Control (1984 $)

Condition	Capital Cost[a]	Annual Chemical ($FeSO_4$) Costs[b]
	$	$/yr
Flow = 3,785 m^3/d $[H_2S]$ = 5 mg/l $FeSO_4$ dose = 23 mg/l	10,000	13,000
Flow = 37,850 m^3/d $[H_2S]$ = 5 mg/l $FeSO_4$ dose = 23 mg/l	21,000	130,000

[a] Includes concrete pad, storage tank, metering pumps, piping, valves, controls, and installation.

[b] Based on typical cost of $FeSO_4$ of $0.67/liter for solution containing 163 g/l $FeSO_4$.

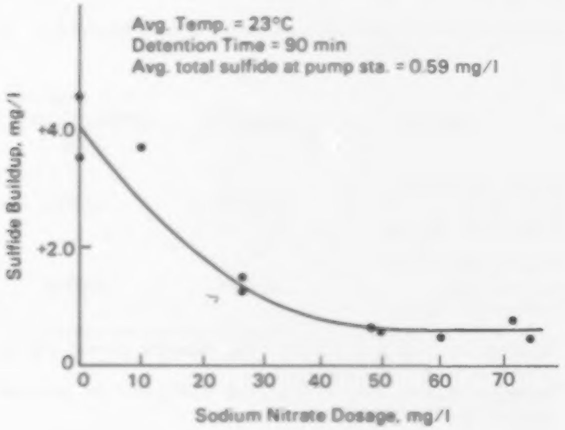
3.3.4 Nitrate

Nitrate can be added as a supplemental oxygen source to control sulfides in wastewater collection systems. Certain bacteria can utilize nitrate as an oxygen source during the biochemical reduction of nitrate to nitrogen gas under anaerobic conditions. Bacteria will utilize hydrogen acceptors preferentially in the order: 1) O_2; 2) NO_3^-; and 3) $SO_4^=$. Thus, if nitrate is present, no sulfate reduction will occur until all of the nitrate has been reduced.

Laboratory tests conducted to show the inhibitory effect of nitrate addition on sulfide production concluded that addition of sodium nitrate to furnish enough oxygen to satisfy 50 percent of the 5-day BOD gave complete protection against odors (44).

Full-scale tests have been conducted on the use of sodium nitrate to control sulfide buildup (3). Several key observations resulted from these tests. First, a population of nitrate-reducing bacteria had to become established in the slime layer. This required several days of nitrate addition before the sewer became "conditioned." Second, nitrate reaction was incomplete, with only about half of the nitrate reacting, the remainder showing up in the force main discharge. Figure 3-27 shows the effect of nitrate dosage on sulfide buildup in a force main. In general, about 10 parts of nitrate were required for every part of sulfide eliminated. At the point of nitrate addition, total sulfide averaged 0.59 mg/l. Before nitrate addition, total sulfide concentration at the force main discharge was 4-5 mg/l. Addition of 27 mg/l of nitrate reduced sulfide in the force main discharge to about 2 mg/l, while dosages of 50 mg/l or greater reduced sulfides to approximately 1 mg/l. It was concluded that nitrate addition has limited utility in controlling sulfides in wastewater collection systems, and may be of practical value only when sulfide concentrations are high (3).

Figure 3-27. Effects of sodium nitrate on sulfide generation in Bluff Cove force main, Los Angeles, CA (3).



Nitrate addition can be effected by a simple low-cost system consisting of a storage vessel, metering pump, piping, valves, and appurtenances. For a solution containing 240 g/l (2 lb/gal) of nitrate, the cost is approximately $0.13/l ($0.50/gal).

Nitrate addition has been successfully used for odor control in anaerobic lagoons, trickling filters, and carbon columns (45-48). Use of nitrate for odor control at existing wastewater treatment facilities is discussed in Chapter 4.

3.3.5 Strong Alkalies

Increasing the pH reduces the proportion of dissolved H_2S in the H_2S - HS^- equilibrium. For example, at a pH of 7.0, equal concentrations of dissolved H_2S and HS^- exist at equilibrium, while at a pH of 8.0, only about 10 percent of the dissolved sulfide exists as H_2S. Since dissolved H_2S is the only form which can be released to the atmosphere, it follows that increasing the pH would reduce odors and corrosion by maintaining the dissolved sulfides in the HS^- form.

Continuous addition of strong alkalies for maintenance of a high pH is generally not practical in collection systems. Tributary flows and production of CO_2 and organic acids from biological action will tend to lower the pH. A drop of 0.5 pH units could result in substantial release of H_2S to the atmosphere.

Both sodium hydroxide and lime have been used for shock dosing of sewers in an attempt to inactivate the sulfide-generating slime layer (3) (14). Figure 3-28 shows the recovery of sulfide buildup capacity with time after shock dosing with sodium hydroxide (NaOH). Depending on the NaOH dosage and the initial pH achieved, times for recovery to 100 percent of normal sulfide buildup ranged from several days to two weeks. The one run at pH 13.2 should not be considered conclusive, since it was not supported by confirmatory observations (3). No detrimental downstream effects were observed with this practice. However, use of large quantities of lime could result in accumulation of calcium carbonate incrustations on the pipe.

Although shock addition of strong alkalies has been shown to be effective in temporary inactivation of the slime layer, continuous addition to prevent H_2S release would not appear to be practical due to potential downstream pH depression from 1) biochemical production of organic acids and CO_2, and 2) tributary flows of normal pH.

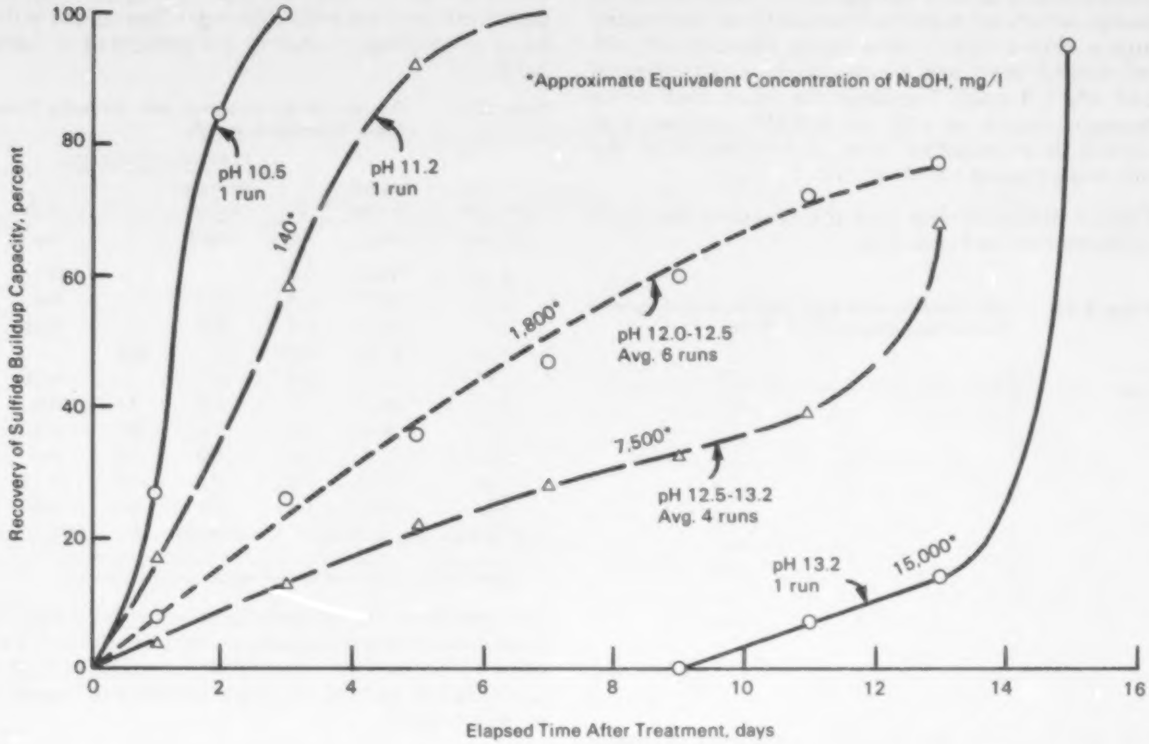
3.3.6 Potassium Permanganate

Potassium permanganate is a strong oxidizing agent, and reacts with H_2S according to the following:

Acidic conditions:

$$3\ H_2S + 2\ KMnO_4 \longrightarrow 3\ S + 2\ H_2O + 2\ KOH + 2\ MnO_2 \quad (3\text{-}15)$$

Figure 3-28. **Impact of shock dosing with NaOH on recovery of sulfide buildup capacity, Los Angeles, CA (3).**



Aklaline
conditions: $3\,H_2S + 8\,KMnO_4 \longrightarrow$
$3\,K_2SO_4 + 2\,H_2O + 2\,KOH + 8\,MnO_2$ (3-16)

Several reactions ranging between these extremes may take place, yielding not only elemental sulfur and/or sulfate, but also thionates, dithionates, and manganese sulfide as possible end products. The actual reaction will depend on specific conditions. Therefore, dosages are case-specific and difficult to predict.

Potassium permanganate has been employed for sulfide control in collection systems and lift stations, generally for small or intermittent applications. In practice, six to seven parts of $KMnO_4$ are required per part of sulfide to be oxidized. The relatively high cost of the chemical, $2.26/kg ($1.03/lb), makes it economically unattractive for continuously treating wastewater streams at high flow rates or wastewater streams that contain substantial concentrations of sulfide. For example, given a flow rate of 378 m^3/d (1 mgd) of wastewater containing 5 mg/l of dissolved sulfide, permanganate requirements based on 6:1 $KMnO_4$:H_2S would be approximately 110 kg (250 lb)/d. Costs for chemicals alone would be in excess of $90,000/year. Metering equipment, however, is simple and easily installed. Total installed cost of a potassium permanganate metering system with a capacity of 45 kg (100 lb)/d is approximately $10,000.

3.4 Case Histories

3.4.1 Force Main Aeration, Gloucester County Utilities Authority (GCUA), Woodbury, New Jersey

For several years, odor problems were noted at the GCUA Wastewater Treatment Plant and at pump stations throughout the collection system. The GCUA undertook an extensive testing program to develop force main aeration design criteria at its Westville pump station, and to establish the cost effectiveness of force main aeration plus H_2O_2 addition compared to the addition of H_2O_2 alone (6)(16).

Analysis of air addition to the Westville force main included development of design parameters, including minimum and maximum detention times; aeration efficiency; pressure relief valve losses; oxygen depletion rates; and operational costs of aeration vs. the addition of hydrogen peroxide.

During the testing program, the average flow to the pump station was 1,385 m^3/d (0.37 mgd). The pump station was originally equipped with constant speed pumps which were later converted to variable speed pumps with a maximum pumping capacity of 5,336 m^3/d (1.41 mgd) and a peak discharge pressure of 510 kPa (74 psig). The Westville force main has a storage volume of 177 m^3 (46,800 gallons) that results in a detention time of 3.12 hours at the minimum flow of 1,870 m^3/d (0.36 mgd).

Oxygen depletion data from the Westville test main are presented in Table 3-22.

Table 3-22. DO Depletion in Raw Wastewater Westville Force Main, Woodbury, NJ (16)

Time	DO
hr	mg/l
0	26.5
1	25.4
2	23.2
3	21.2
4	19.1
5	15.3
6	8.1
7	0.3
8	0

The design air supply rate for the Westville pump station was based on respirometric data that indicated an oxygen demand of 7-12 mg/l-hr, which resulted in a design injection rate of 37.4 mg/l. The air compressor was sized to deliver a maximum of 50 mg/l DO with an assumed 40-percent oxygen transfer efficiency within the force main. This resulted in a design compressor capacity of 0.44 m^3 (15.4 cu ft)/min. For the Westville pump station, dual 3-HP 830 rpm compressors were selected, each with a capacity of 0.25 to 0.28 m^3 (9 to 10 cu ft)/min at a receiver pressure of 655 kPa (95 psig). Each compressor had a duty cycle of 0.49.

In the Westville installation, the compressor controls were installed for both start-stop and dual-mode operation. In the start-stop cycle, the compressor operates only when the pump is on. In the dual-mode cycle, the compressor operates based on a low-pressure switch in the receiver tank. When the receiver high-pressure switch is reached, the compressor continues to operate if the pump is on until the pump cycle is over. If the pump is not on, the compressor shuts down after operating for a preset time. Time delays are also provided in this design to delay air injection after pump starting.

The air injection rates for the Westville installation were 0.08 to 0.4 m^3 (3 to 14 cu ft)/min. Without aeration, the Westville force main discharged a dark odorous wastewater with a dissolved sulfide concentration of approximately 0.6 mg/l. Background sulfate concentrations were 40 to 80 mg/l. The results of the force main aeration testing are presented in Table 3-23.

Table 3-23. Results of Air Injection into Westville Force Main, Woodbury, NJ (6)

		Force Main Discharge			
Air Feed	Oxygen Injected	DO	Dissolved* Sulfide	pH	H_2S
m^3/min	mg/l	mg/l	mg/l		mg/l
0.08	18.0	0.0	-	-	0.1
0.10	21.0	0.7	1.3	-	0.6
0.11	24.0	0.0	0.6	-	0.26
0.13	27.0	0.15	-	6.8	-
0.14	30.0	0.0	0.2	-	0.09
0.14	30.0	-	0.0	7.5	0.0
0.17	36.0	1.0	0.4	6.7	0.2
0.22	47.0	1.0	0.0	6.5	0.0
0.23	50.0	1.4	0.2	6.5	0.12
0.24	50.0	1.6	0.1	6.7	0.05
0.20 to 0.40	42.0 to 84.0	-	0.3 to 3.0	6.8	0.13 to 3.0

*Concentrations prior to aeration were typically 0.6 mg/l.

The results of the preceding tests indicate that air injection into the force main at rates of 0.08 to 0.4 m^3 (3 to 14 cu ft)/min successfully reduced sulfide to levels as low as 0.05 mg/l, and provided DO levels of up to 1.6 mg/l.

The average design aeration rate for the Westville force main was 0.17 m^3 (6 cu ft)/min, which was approximately three times the theoretical aeration rate based on the oxygen demand of the wastewater. This was due to mixing efficiency in the force main and the loss of air through air relief valves. In the Westville force main, air losses through the relief valves were determined to be 30 percent with the valves operating at 414 kPa (60 psig) and a receiver pressure of 655 kPa (95 psig) (16).

The installed cost of the Westville force main aeration system was $13,500 (1979) with an estimated operation and maintenance cost of $750/year. For a 20-year design lifetime and an interest rate of 8 percent, the total amortized cost was $3,025/year. The use of air injection was found to reduce hydrogen peroxide addition by 70 percent, which amounted to an annual savings of $7,770 in hydrogen peroxide costs.

A comparison of the annual costs for peroxide addition alone and peroxide addition with force main aeration is presented in Table 3-24 for the Westville pump station. This comparison indicates a net savings of $4,780/yr for force main aeration plus hydrogen peroxide addition over hydrogen peroxide addition alone at the Westville pump station.

Table 3-24. **Costs for Sulfide Control in Westville Force Main Using H_2O_2 and H_2O_2 with Air Injection, Woodbury, NJ (6)**

H_2O_2		Aeration			Total	
Without Aeration[a]	With Aeration[a]	Capital[b]	Electrical[c]	Maintenance	Without Aeration	With Aeration
$/yr		$/yr			$/yr	
11,100	3,330	1,375	865	750	11,100	6,320

[a] H_2O_2 @ $0.083/kg.

[b] Amortized capital cost assuming installed cost of $13,500 (1979), 20-year life, 8-percent interest.

[c] Electrical energy cost = $0.04/kWh.

3.4.2 Oxygen Injection, Delta Diablo Sanitation District 7A, Antioch, California

In 1976, an Environmental Impact Report compiled by the East/Central Contra Costa County Wastewater Management Agency concluded that the existing treatment facilities for the cities of Pittsburg and Antioch, California, and the unincorporated community of West Pittsburg, California, could not meet EPA discharge requirements, and that a new regional facility would be required. Design was begun in 1976, and the new treatment facility went on-line in 1982. The three existing plants were abandoned, and wastewater from these communities was conveyed through a series of new force mains to the new 36,000-m^3/d (9.5-mgd) trickling-filter, activated-sludge secondary treatment plant (49).

Because the length of the force mains was as much as 11.3 km (7 miles), substantial sulfide generation was anticipated. Evaluation of sulfide control alternatives led to the selection of pure oxygen injection into the force mains at three locations to maintain sufficient dissolved oxygen to prevent generation of sulfides. Figure 3-29 is a map of the sewer service area showing locations of the pump stations where pure oxygen is injected into the force mains. The new force mains are coated steel, cathodically protected to minimize electrochemical corrosion. Pipe sizes and wastewater flows for the three force mains where oxygen is injected are presented in Table 3-25.

Figure 3-29. **Sewer service area, Delta Diablo Sanitation District 7A, California.**

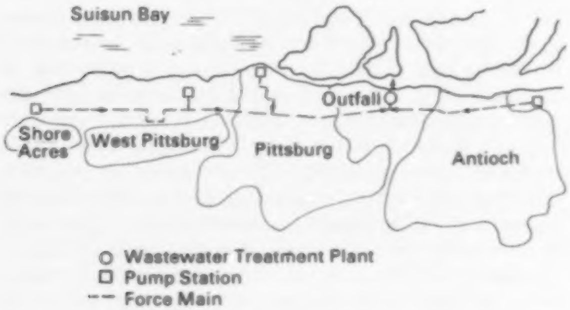


Table 3-25. **Force Main Characteristics at Pure Oxygen Injection Points, Antioch, CA**

Location	Pipe Diameter	Design Flow	Actual Flow
	cm	m^3/d	m^3/d
Antioch	61	20,820	15,140
Pittsburg	61	32,360	7,570
Shore Acres	41	14,570	1,890

Total flows to the plant were approximately 30,280 m^3/d (8.0 mgd) in the winter and 28,390 m^3/d (7.5 mgd) in the summer of 1983. The plant is presently being expanded to a capacity of 47,690 m^3/d (12.6 mgd). The target level of dissolved sulfide entering the plant is 0.1 mg/l.

Total sulfides entering the plant before oxygen injection was initiated were 3.0 to 6.0 mg/l. Dissolved sulfides were typically 1.5 to 5.0 mg/l.

Oxygen injection rates are automatically varied in proportion to the flow. At the Pittsburg pump station oxygen is injected at the discharge side of the pump at an average rate of 490 kg (1,080 lb)/d. The oxygen feed system went on-line in October, 1981. At the Antioch pump station, average oxygen injection rates are 345 kg (760 lb)/d in the winter and 605 kg (1,330 lb)/d in the summer. The oxygen injection system was commissioned in 1979. Because of the low flows in the Shore Acres force main, refilling of the pure oxygen storage tank has not yet been required since commissioning in October 1981. Thus, the tank has not been calibrated to allow estimation of oxygen injection rates at that site.

Raw wastewater entering the plant is analyzed for sulfides on the average of three times per week. If sulfides begin to exceed target levels of 0.1 mg/l, two options can be initiated—increase the oxygen dosage or clean the line of accumulated slimes by the use of a "pig." Pig launch sites were designed into the force main system to facilitate line cleaning. In the Antioch force main, actual oxygen requirements to maintain aerobic conditions sometimes exceed the capacity of the oxygen injection unit, particularly in the summer. Periodic cleaning of the line (as much as once every 3 weeks in the summer) has been found to be very effective in reducing sulfides to acceptable levels at the end of the main. This procedure requires approximately 4 hours of operational staff time per cleaning.

Oxygen injection into the three force mains in the Delta Diablo Sanitation District has been successful in controlling sulfides in the collection system and at the headworks of the treatment plant. Total sulfides are 0.0 to 1.5 mg/l at manholes just upstream from the headworks. Dissolved sulfides entering the plant are 0.0 to 0.7 mg/l during oxygen injection. Typical dissolved H_2S (gas phase) levels are 0.0 to 0.1, depending on pH.

Total installed cost for the three oxygen injection systems was approximately $18,000, broken down as follows: Antioch—$8,000 (1979), Pittsburg—$5,000 (1981), and Shore Acres—$5,000 (1981). As the oxygen storage vessel and evaporator are leased from oxygen suppliers, these costs include concrete pad, piping, valves, controls, appurtenances, and installation. Annual operational costs for the three systems amounted to approximately $31,200 in 1983. Of this figure, about $20,000 was expended for rental of equipment from the oxygen supplier, the remainder for purchase and delivery of liquid oxygen. Any maintenance problems with the oxygen storage vessel and evaporator are the responsibility of the oxygen supplier. According to plant operational staff, any such problems are promptly attended to by the supplier.

Occasionally, odors are generated from the four trickling-filter towers. These units are 6.4-m (21-ft) deep employing modular plastic media. Forced draft ventilation by reversible fans was included in the design. Under normal conditions, the trickling filters are naturally ventilated. However, during certain times of the year when the temperature differential between the ambient air and the wastewater is insufficient to induce a natural draft, the ventilating fans are brought on-line in an attempt to control odor emissions. Air quality surrounding the plant and pump stations in the collection systems is monitored regularly by the State Air Quality Regulatory Agency. A planned residential housing development in close proximity to the plant may require greater control of odor emissions in the future.

3.4.3 Hydrogen Peroxide Addition, Palm Beach County, Florida

In 1979, the county of Palm Beach, Florida, began operation of new wastewater transmission facilities and regional treatment plant to serve the east-central region of the county. The collection system consisted of 39 km (42 mi) of force mains, ranging in diameter from 10 cm (4 in) to 91 cm (36 in), and 25 lift stations.

Almost immediately upon commissioning the system, the county began experiencing severe concentrations of H_2S around the pump stations which caused numerous citizen complaints. Attempts at controlling sulfide included chlorine addition, air injection, and ferrous sulfate addition. None of these techniques proved satisfactory for the reasons cited (34):

- Chlorine - reacted too fast, odor problems
- Air - poor dissolution, air binding
- Ferrous sulfate - did not provide needed residual DO

The failure of the preceding alternatives to provide satisfactory H_2S control, in conjunction with court action by affected citizens, led the county to investigate use of hydrogen peroxide for sulfide control.

A diagram of the key components of the collection system is presented in Figure 3-30. This figure indicates length and diameters of force mains, location of pumping stations, and locations of H_2O_2 dosing stations. In general, dosing stations were located 20 to 40 min (wastewater travel time) upstream of the respective pump stations to provide sufficient reaction time.

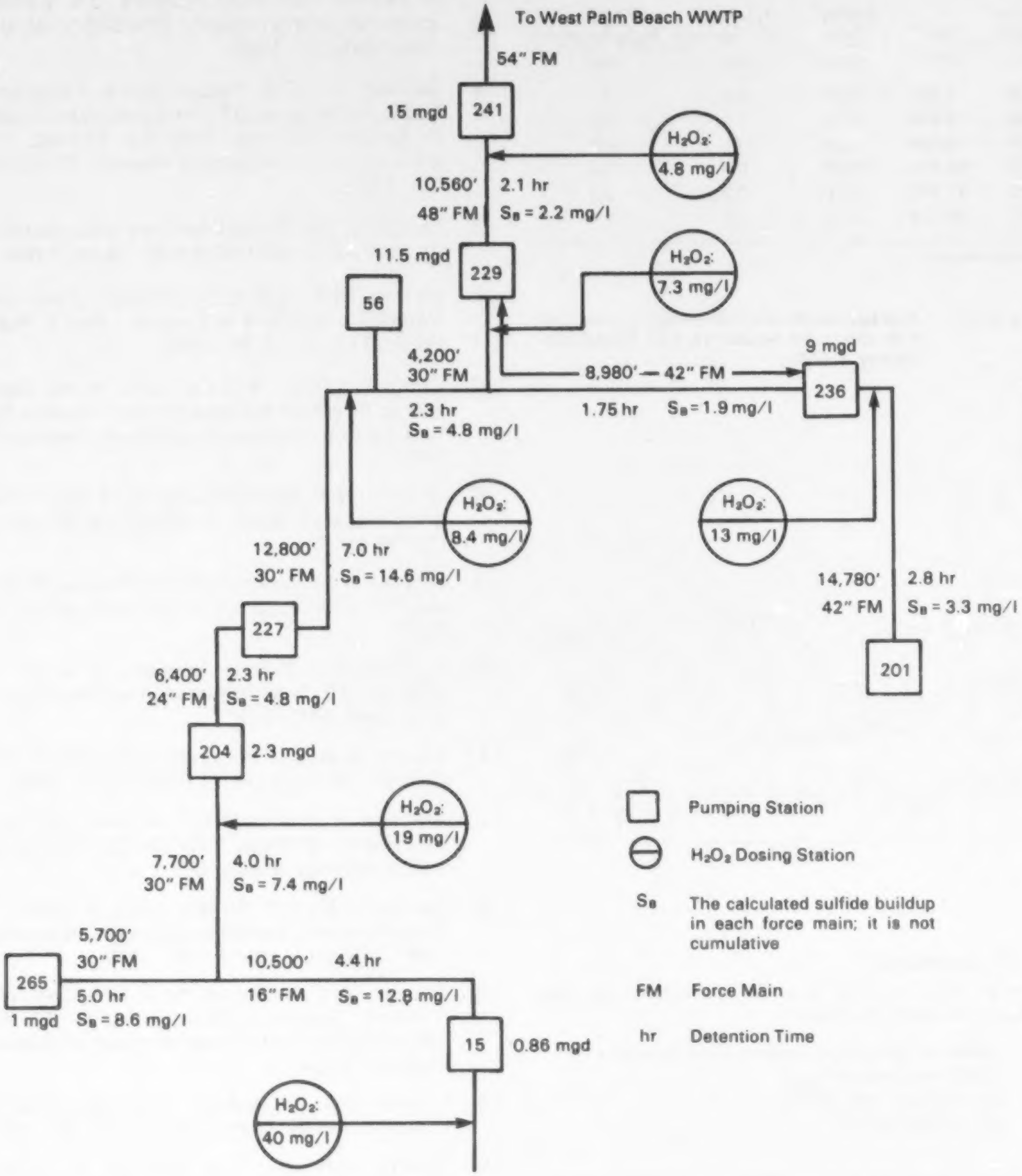
Dates of commissioning of the H_2O_2 dosing stations are shown below:

Pump Station	Commissioning Date
241	June,1980
229	June, 1980
236	June, 1981
15	August, 1982
204	December, 1982
56	December, 1982

Few data are available indicating H_2S levels prior to H_2O_2 dosing. Figure 3-30 presents calculated sulfide buildup in each force main reach using the Pomeroy and Parkhurst predictive equation, assuming an average annual wastewater temperature of 28°C (82°F) and an average BOD of 150 mg/l. Generally, predicated sulfide levels were in the range of 2 to 15 mg/l.

Table 3-26 summarizes the average wastewater flows and H_2O_2 dosing rates at each of the six pump stations, and gives measured H_2S concentrations at these locations after addition of H_2O_2. As can be seen from the data, injection of H_2O_2 was effective in reducing H_2S concentrations in the wastewater to below 0.5 mg/l at all six pump stations. In addition, further sulfide generation was inhibited by maintaining DO levels greater than 1.0 mg/l. Table 3-26 indicates average annual dosing rates. In practice, dosages were varied depending on the rate of sulfide generation as affected by temperature, flow rates, wastewater characteristics, etc. Figure 3-31 shows wastewater flows and temperatures and H_2O_2 usage from January, 1982 through March, 1983 (50). Note

Figure 3-30. Wastewater collection system with H_2O_2 dosing stations, Palm Beach County, FL.


To West Palm Beach WWTP
54" FM
15 mgd
241
H₂O₂: 4.8 mg/l
10,560'
48" FM
2.1 hr
Sₐ = 2.2 mg/l
11.5 mgd
229
H₂O₂: 7.3 mg/l
56
4,200'
30" FM
2.3 hr
Sₐ = 4.8 mg/l
9 mgd
8,980' — 42" FM
236
1.75 hr
Sₐ = 1.9 mg/l
H₂O₂: 8.4 mg/l
H₂O₂: 13 mg/l
12,800'
30" FM
7.0 hr
Sₐ = 14.6 mg/l
14,780'
42" FM
2.8 hr
Sₐ = 3.3 mg/l
227
201
6,400'
24" FM
2.3 hr
Sₐ = 4.8 mg/l
204
2.3 mgd
H₂O₂: 19 mg/l
7,700'
30" FM
4.0 hr
Sₐ = 7.4 mg/l
5,700'
30" FM
265
5.0 hr
1 mgd
Sₐ = 8.6 mg/l
10,500'
16" FM
4.4 hr
Sₐ = 12.8 mg/l
15
0.86 mgd
H₂O₂: 40 mg/l
Pumping Station
H₂O₂ Dosing Station
Sₐ The calculated sulfide buildup in each force main; it is not cumulative
FM Force Main
hr Detention Time

that the greatest rate of H_2O_2 usage generally corresponds to periods of high temperatures and low wastewater flows.

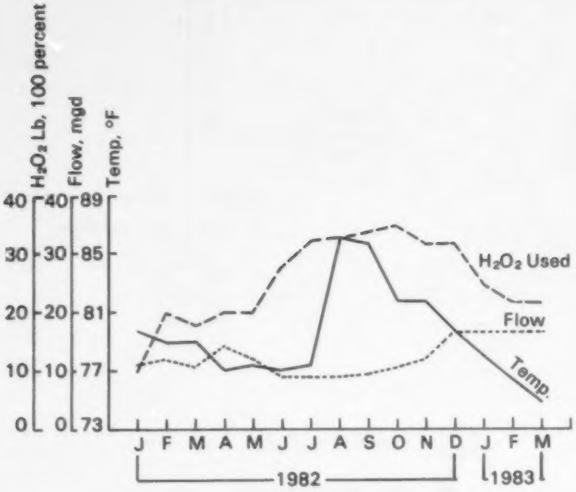
The first dosing stations were temporary installations, and were commissioned under a leasing arrangement with a supplier by which the cost of the dosing stations was incorporated into the unit cost of purchased H_2O_2. This cost also covered all start-up and monitoring costs by the supplier. Palm Beach County has since constructed permanent dosing stations.

Table 3-26. **Effectiveness of H_2O_2 for Sulfide Control at Palm Beach County, FL (50)**

Pump Station	Flow*	Dosing* Rate	H_2S After* H_2O_2 Addition	DO After* H_2O_2 Addition
	m³/d	mg/l	mg/l	mg/l
15	1,890	40.0	0.5	3.3
204	6,430	19.0	0.4	3.4
56	6,430	8.4	0.1	4.5
236	33,310	13.0	0.4	1.4
229	41,640	7.3	0.2	2.3
241	45,420	4.8	0.1	1.4

*Average values.

Figure 3-31. **Average wastewater temperature, flow and H_2O_2 used at Lift Station No. 229, Palm Beach County, FL (50).**



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National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650

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Chapter 4

Odor and Corrosion Control in Existing Wastewater Treatment Plants

4.1 Introduction

Odor and corrosion are significant problems at many wastewater treatment facilities. In a 1973 survey of 500 treatment plants in the United States, 40 percent of the plant superintendents responded that they had received complaints about odors (1). In addition, superintendents at 37 percent of these plants indicated that odor abatement measures, either through process modification or installation of odor control equipment, have been, or were planned to be, instituted.

Corrosion of concrete and steel in wastewater treatment plants can result in significant maintenance and replacement costs over the lifetime of the plant. Corrosive substances in purely domestic wastewaters are principally H_2S, chlorine, aggressive water, ammonia, and salt (2). Industrial wastes containing acids, alkalies, and various organic chemicals discharged into a municipal sewer system may aggravate the problem.

This chapter describes likely sources of odors at wastewater treatment plants, alternatives for control of odor emissions, techniques for treatment of odorous air, odor masking and counteraction agents. In addition, a section on corrosion addresses the mechanisms of corrosion of concrete and metals, corrosion control alternatives, and selection of corrosion-resistant coatings and materials.

4.2 Sources of Odors in Wastewater Treatment Plants

Most unit processes in wastewater treatment plants are potential sources of odor. This section will discuss, in flow chronology, each unit process and the conditions conducive to odor generation.

4.2.1 Headworks and Preliminary Treatment

If odorous gases such as H_2S are dissolved in wastewater entering the headworks of a wastewater treatment plant, turbulence induced by drops, flumes, aerated grit chambers, or similar structures will cause the gases to be released from solution. High organic strength septic sidestreams from sludge processing operations, such as wet oxidation decant liquors, filtrates, digester supernatants, filtrates and centrates, may also release malodorous gases under such conditions.

Because of long detention times, flow equalization basins may become septic if not aerated. Inadequate mixing in such basins may also result in increased deposition of organic material, which can aggravate the problem.

Accumulation of organic debris in influent channels, on bar screens, comminutors, and fine screening devices can result in odor generation if regular cleaning and flushing is not practiced. Grit chambers and grit conveyance systems can also be serious sources of odors due to the organic coating on grit particles. This is especially true in smaller plants where grit may be stored for long periods of time before disposal (3).

4.2.2 Primary Clarifiers

Primary clarifiers can be a source of odors if improperly designed and maintained. If scum removal mechanisms are inadequate, resulting scum accumulation and subsequent putrefaction will result in odor generation. Infrequent or incomplete withdrawal of settled solids can result in septic conditions and generation of odorous gases that can also result in sludge rising to the surface due to buoyancy from trapped gases (3). Discharge over the effluent weirs can release odorous gases dissolved in the primary effluent.

4.2.3 Fixed Film Reactors

Fixed film reactors, such as trickling filters and rotating biological contactors, can be sources of odors when the air supply to the biological film is inadequate. This often occurs during hydraulic overload conditions. Plugging or improper sizing of underdrains in trickling filters reduces air circulation and promotes anoxic conditions. Poor distribution of the wastewater onto the media results in discontinuous wetting and excessive slime buildup, which can lead to increased presence of anaerobic zones and subsequent odor generation.

4.2.4 Activated Sludge Basins

Aeration basins are not normally significant sources of objectionable odor. However, the existence of poor mixing characteristics can result in deposition of organic solids in corners or along the edges of the tank (3). Such sludge deposits may generate odorous gases at a rate greater than they can be oxidized by the overlying aerated liquid. Clogging of diffusers causes uneven distribution of air and may result in anoxic zones and solids deposition. Aeration tank walls that are intermittently wetted by wastewater spray may develop putrescible slimes that can generate odors.

4.2.5 Final Clarifiers

Final clarifiers are not normally sources of odors if upstream aerobic stabilization processes are properly designed and operated. The major consideration in preventing odor generation in clarifiers is maintaining adequate rates of sludge withdrawal to prevent septic conditions. In addition to being a source of foul odors, septic sludge will create additional oxygen demand when returned to the aeration basins.

4.2.6 Sludge Thickening, Conditioning, and Holding

Sludge handling systems are normally the most significant source of odors in wastewater treatment plants. Unit processes which allow exposure of the sludge to the atmosphere, such as holding tanks and thickeners, will generate odors with intensities ranging from mildly offensive to nauseating. Virtually all sludges emit odors, but fresher sludges generate less intense, less offensive odors. Septic sludges emit highly offensive and persistent odors. Sludge thickeners are often the cause of odor complaints from neighborhoods surrounding municipal wastewater treatment plants due to exposure of raw sludge to the atmosphere (4). Wet oxidation processes operated at temperature and pressure regimes needed for conditioning sludges are a major source of odors unless special precautions have been taken to contain and treat the odorous discharges. One of the major sources of odor from this process is decant tanks that are generally located outside of the housed sludge handling facilities and are often uncovered.

4.2.7 Sludge Dewatering, Stabilization, and Storage

Sludge dewatering processes are often sources of odors. Such processes include vacuum filtration, plant and frame and moving belt pressure filtration, centrifugation, and gravity and vacuum drying beds. The extent of odor generation will vary depending on the type and characteristics of the sludge, the method used for dewatering, and the chemicals used for conditioning. For example, vacuum filtration of a high pH digested sludge will often result in release of ammonia.

Sludge stabilization processes include anaerobic digestion, aerobic digestion, lime stabilization, composting, and chlorine oxidation. In most cases, odors generated during sludge stabilization are not highly offensive if the system is properly designed and operated. "Sour" anaerobic digesters and overloaded aerobic digesters, however, will often generate offensive odors. Lime stabilization processes may generate large quantities of ammonia gas resulting from the high pH. This has been noted to be a serious problem in physical-chemical treatment plants employing lime precipitation. Properly designed aerated pile composting systems exhaust odorous air through piles of finished compost. Windrow composting systems may generate significant odors during turning of the piles, particularly if insufficient quantities of bulking agent are employed to allow proper air circulation. Properly operated chlorine oxidation systems will generate medicinal or chlorine odors. However, insufficient chlorine dosages or long-term storage of the chlorinated sludge may result in putrefaction and release of objectionable odors.

Sludge storage tanks, basins, and lagoons are principal sources of odor at wastewater treatment facilities. The problem is difficult to control since storage vessels are often uncovered, and the large surface areas provide high exposure of the sludge to the atmosphere. Wind action on the surface of storage lagoons can compound the problem.

4.2.8 Sludge Incineration and Solids Reduction

Processes included in this category include multiple hearth and fluidized bed incineration, starved air combustion, pyrolysis, wet oxidation and flash drying. Odor problems in combustion processes result from incomplete oxidation of odorous gases, or from spillages during sludge transfer operations. In many cases, gas scrubbers or direct flame oxidation systems employed to meet air pollution emission control requirements are effective in reducing odorous discharges.

4.2.9 Process Sidestreams

Liquid streams resulting from sludge processing operations have a variety of names depending on the type of process they arise from and include supernatants, centrates, filtrates, elutriates, thermal process decant liquors, and filter backwash waters. In many cases, these sidestreams exhibit very high COD, BOD and ammonia-nitrogen concentrations and are often major sources of odor.

Sidestreams are often returned directly to the headworks of the plant where they may cause odor problems due to turbulence and release of odorous gases, or due to BOD overloading and rapid depletion of DO. In many instances, high strength sidestreams require pretreatment before return to the wastewater treatment processes.

4.2.10 Septage Handling

Septage receiving and handling facilities are major sources of odor at municipal wastewater treatment plants, since these wastes are almost always septic and are often handled on a periodic and unscheduled basis. Septage is composed of highly putrescible materials which cause very persistent and objectionable odors. Odors are often generated during transfer from septage hauling trucks to holding tanks at the plant site and during discharge to manholes, lift stations, headworks or sludge processing facilities. Turbulent conditions result in release of odorous gases to the atmosphere. Uncontrolled addition of septage to the main wastewater stream may also result in rapid DO depletion and subsequent odor generation.

4.2.11 Physical-Chemical Treatment Plants

Odor generation and corrosion problems have been noted to be especially severe in physical-chemical treatment systems, since these systems provide little, if any, opportunity for oxidation of dissolved sulfides thay may be present in the influent waste stream. The problem is compounded, especially in the smaller facilities, because the unit processes such as pretreatment, lime clarification, filtration and carbon adsorption are often housed, thus creating a confined atmosphere that extends the corrosive influence beyond the original source. Single or two-stage lime precipitation may reduce sulfide release potential, but can liberate ammonia, which is especially corrosive to instrumentation and control systems. Carbon adsorption systems often become anoxic and release significant amounts of H_2S when opened to the atmosphere, such as during backwashing and carbon transfer operations.

4.3 Control of Odors in Existing Wastewater Treatment Plants

There are three general categories of odor control:

- Prevention of odorous emissions
- Collection and treatment of odorous air
- Odor modification, counteraction and masking

In many cases, odor emissions can be reduced or eliminated through improved operation and maintenance practices. Regular, frequent cleaning of preliminary treatment devices such as comminutors, bar screens, and grit chambers; flushing of tank walls; removal of sludge deposits from influent and interprocess channels; and increased rate of withdrawal of settled solids are examples of routine operation and maintenance techniques that are necessary to control odors. Where high sulfide concentrations are present in the influent wastewater, treatment in the collection system through the injection of air or oxygen, or the addition of chemicals such as hydrogen peroxide or metal salts, can reduce or eliminate odors at the headworks. Addition of chemicals, such as sodium nitrate, at the headworks can also be effective for odor control in follow-on unit processes, such as trickling filters. Potassium permanganate and hydrogen peroxide have been used effectively for odor control in sludge processing operations (e.g., dewatering). Odors from many wastewater treatment operations are released due to air stripping, which can be reduced by subsurface discharge of liquids such as sidestreams and septage, which contain dissolved odorous compounds.

Where odors are generated in enclosed spaces, such as sludge processing buildings, covered holding tanks, and wet wells, the odorous air can be effectively treated prior to release to the atmosphere by a variety of techniques, including wet scrubbers, activated carbon, chemical adsorbers, and soil or compost filters.

Finally, odors can be made less objectionable through the use of odor masking and counteractive agents. Since this often involves merely replacing an objectionable odor with a more pleasant one, this approach is generally the least preferred of the available techniques for odor control and should not be considered for a permanent solution.

4.3.1 Prevention of Odorous Emissions

4.3.1.1 Operation and Maintenance

Good housekeeping is always essential to the prevention of odors being generated. Many odors associated with wastewater treatment operations can be controlled or eliminated by ensuring that process components are kept clean and free of accumulated grease, solids and debris.

Bar screens and preliminary treatment processes should be cleaned daily to remove any accumulated organic debris that can putrify and cause odors. Grit and screening conveyance systems should be flushed with water to remove organic debris and grit, and the materials should be transferred to closed containers to minimize escape of odors.

Scum scrapers, pits and wet wells on primary clarifiers should be cleaned frequently and chemically treated, if necessary, to remove accumulated grease and scum and reduce the potential for biological degradation. Scum and grease collection wells and troughs should be emptied and flushed regularly to prevent putrefaction of accumulated organics. Settled solids should not be allowed to accumulate in the bottom of clarifiers for long periods, since septic conditions can develop within 2 to 6 hours. Pumping frequency should, therefore, be adjusted to prevent settled solids from being detained for more than 1.5 to 2 hours (3).

Trickling filter media should be continuously wetted and kept free from plugging to allow adequate air circulation for the fixed film to remain aerobic and odor free. During periods of low flow, this can be accomplished by increasing recirculation rates. Distribution nozzles should be kept clear at all times to allow uniform application of wastewater to the media. Filter underdrains and drain lines should be checked periodically to ensure that they are not plugged, thereby reducing air circulation through the media which causes anaerobic conditions and odor generation (5). Interior walls of trickling filters and walls surrounding rotating biological contactors should be cleaned and flushed regularly.

In activated sludge basins, sufficient and complete mixing is essential to prevent deposition of solids. Clogging of diffusers results in poor mixing, and is manifested by a lack of turbulence and an accumulation of foam, bubbles, or scum at the surface. Air piping and diffusers should be inspected and cleaned periodically. Tank walls subjected to intermittent wastewater spray should be cleaned regularly.

Scum scrapers, troughs, weirs and interior walls of final clarifiers should be cleaned and flushed regularly to remove putrescible organics. Settled solids should not be allowed to accumulate long enough for anaerobic conditions and the resultant odor generation to begin.

Since wastewater sludge is a significant source of odors, special care should be taken to ensure that sludge transfer systems such as bucket conveyors, screw pumps, belt conveyors, and conduits be kept as clean as possible. Spillages should be cleaned and flushed immediately to prevent unnecessary odor generation. Elutriation water or dilution water for gravity thickeners should contain maximum DO to aid in odor control. Sludge blankets in flotation thickeners should be removed at regular frequent intervals or continuously.

Gas from poorly operating anaerobic digesters is odorous and, if possible, should be burned in the waste gas burner. An auxiliary fuel source may sometimes be necessary to ensure complete oxidation.

Septage received at wastewater treatment plants should be transferred from the hauler truck into a closed tank or subsurface receiving basin by using quick-disconnect, watertight fittings. This prevents splashing, turbulence, and release of odors. Spills should be immediately flushed with water. Provision should be made for control of the rate of addition of the septage into the wastewater stream to avoid excessive DO depletion. Allowable loadings to mainstream processes will depend on the aeration and solids handling capacity of the plant and the characteristics of the septage. One reasonable guideline is to limit the volatile solids loading from the septage to 10 percent of the volatile solids entering in the raw wastewater over the same time period.

Extraneous odor generation in wastewater treatment plants can be minimized through a regular inspection and maintenance program that involves frequent removal of accumulated solids and organic debris, and regular cleaning of tanks, unit process equipment, and hardware that come in contact with wastewater or wastewater sludges. Such a program can usually be implemented at little cost, often with substantial reduction of odor generation.

4.3.1.2 Upstream Treatment

Odor problems at wastewater treatment plants are often caused by the release of H_2S gas at the headworks. If this is the case, it is likely that odors are being released in the collection system as well. Rather than attempt to collect and treat the odorous gas at each point where it is being released, it is often more cost effective to control the sulfides by one or more of the techniques discussed in Chapter 3. These include:

1. Air injection or entrainment
2. Pure oxygen injection
3. Chemical addition
    - chlorine
    - hydrogen peroxide
    - metal salts
    - nitrates

The point of injection gas or chemical addition is dependent on how far upstream odor control is desired, and the reaction time required to minimize or eliminate odors at the plant headworks. In many cases, multiple injection points are necessary. For a detailed discussion of in-sewer odor control, the reader is referred to Chapter 3.

4.3.1.3 Chemical Addition

Direct chemical addition to wastewater, wastewater sludge, or process sidestreams can be a simple and effective technique for odor control. Chemicals used for this purpose include hydrogen peroxide, potassium permanganate, sodium nitrate, and chlorine.

a. Hydrogen Peroxide

Under conditions typically found in municipal wastewater, hydrogen peroxide reacts with H_2S according to the following reaction:

$$H_2O_2 + H_2S \longrightarrow S + 2\,H_2O \qquad (4\text{-}1)$$

In addition to the oxidizing sulfide, excess hydrogen peroxide decomposes to yield oxygen and water. Thus, it has the added benefit of increasing the DO of the stream. Use of hydrogen peroxide for sulfide control in collection systems has been described in Chapter 3. If odor emissions from manholes, lift stations, etc. are not a problem, but the wastewater entering the plant is high in dissolved sulfides, hydrogen peroxide can be injected upstream of the headworks to minimize or prevent odor generation when the wastewater enters the plant. An injection point should be selected that will provide 15 to 45 minutes reaction time, depending on H_2S concentration and wastewater characteristics.

A similar application for odor control with H_2O_2 has been injection upstream of the primary clarifiers. Long wastewater residence times in the clarifiers due to operation at flows significantly less than design flows, or inadequate sludge withdrawal rates, can result in substantial sulfide generation in the clarifiers. Rapid DO depletion problems may also result from return of anaerobic sidestreams to the headworks. Turbulence promoted by the fall of the wastewater from the effluent weirs into the collection trough can release H_2S, resulting in odor generation.

In 1982, the city of Baltimore, Maryland conducted a demonstration using H_2O_2 for control of H_2S being released at the primary effluent weirs at the 674,000 m^3/d (178 mgd) Back River Wastewater Treatment Plant. Table 4-1 shows the results of the demonstration (6). In comparing the data for H_2S concentrations before and after the primary effluent weirs prior to H_2O_2 dosing, it can be seen that substantial H_2S losses were occurring during discharge over the weirs into the effluent trough. This was attributed to stripping of the H_2S from solution (6). Average mass ratio of applied H_2O_2 to H_2S at the dosing manhole, located 150 m upstream of the headworks, was 2.14:1 in order to oxidize existing sulfides and to prevent additional sulfide formation. The data in Table 4-1 show that maximum H_2S levels before the primary weirs were reduced from 2.5 to 1.0 mg/l. Prior to H_2O_2 addition, an average of 0.32 mq/l of H_2S was being released from solution during discharge over the primary weirs, but after H_2O_2 addition, an average of only 0.11 mg/l H_2S was being released.

Hydrogen peroxide has also been employed for odor control in sludge handling systems (7-9). At the 757,000-m^3/d (200-mgd) secondary treatment plant operated by the Allegheny County Sanitary Authority (Pittsburgh, PA), major odor problems occurred in the sludge handling system (9). The unit processes of concern were the constant head tanks, the sludge holding/mixing tanks, and the vacuum filters. Air from the vacuum filter building was exhausted through an activated carbon system for odor control. However, odors generated and released in the building were greater than anticipated and far beyond the capacity of the carbon filters. Trials were conducted with H_2O_2 addition to the constant head tanks. Results are shown in Table 4-2. A dosage of 40 ppm to the sludge was sufficient to reduce atmospheric H_2S levels above the mixing tanks and in the building exhaust to zero, and to reduce dissolved sulfide concentrations in the vacuum filter filtrate to 0.1 mg/l. Based on these trails, a permanent system was

Table 4-2. **Use of H_2O_2 for H_2S Odor Control in Sludge Handling System, Pittsburgh, PA (9)**

	Atmospheric H_2S		Dissolved Sulfides
H_2O_2 Dose	Over Mix Tank	Building Exhaust Duct*	Vacuum Filter Filtrate
ppm	ppm		ppm
0	8	2	1 to 30
	2	2	1 to 50
20	0	0	0.35
	0	0	0.25
	0	0	0.25
40	0	0	0.10
50	0	0	0.10
	0	0	0.10

*Carbon filters were in use at all times for treatment of exhaust air, but were not completely effective, as shown by data collected prior to H_2O_2 injection.

Table 4-1. **Use of H_2O_2 to Control H_2S Odors at Baltimore, MD (6)**

	H_2S Concentration					
	Without H_2O_2			With H_2O_2[a]		
Location	Maximum	Minimum	Average	Maximum	Minimum	Average
		mg/l			mg/l	
Dosing Manhole[b]	1.8	0.3	0.92	NR[c]	NR	NR
Before Primary Weirs	2.5	0.2	0.96	1.0	0.2	0.55
After Primary Weirs	0.9	0.4	0.64	1.0	0.1	0.44

[a] H_2O_2 dosing schedule: 8 AM - 8 PM: 2.8 mg/l
8 PM - 8 AM: 1.1 mg/l

[b] Located approximately 150 m (500 ft) upstream of headworks.

[c] Data not reported.

constructed for injection of 40 ppm of H_2O_2 at a point 46 m (150 ft) upstream of the constant head tanks (9).

b. Potassium Permanganate

As discussed in Chapter 3, potassium permanganate ($KMnO_4$) is a strong oxidizing agent which reacts with H_2S as follows:

Acidic conditions:
$$3\,H_2S + 2\,KMnO_4 \longrightarrow 3\,S + 2\,H_2O + 2\,KOH + 2\,MnO_2 \qquad (4\text{-}2)$$

Alkaline conditions:
$$3\,H_2S + 8\,KMnO_4 \longrightarrow 3\,K_2SO_4 + 2\,H_2O + 2\,KOH + 8\,MnO_2 \qquad (4\text{-}3)$$

$KMnO_4$ reacts with many odor-producing compounds, including aliphatic, aromatic, nitrogen-containing, sulfur-containing, and inorganic compounds. However certain compounds in these categories do not react readily with $KMnO_4$. For lists of compounds which do and do not react with $KMnO_4$, the reader is referred to References 3 and 10.

$KMnO_4$ has been applied to various points in the liquid stream of a wastewater treatment plant as well as to sludge processing operations such as dewatering. Required $KMnO_4$:H_2S weight ratios to achieve sulfide control in wastewater generally range from 2.5:1 to 6:1. Figure 4-1 shows percent sulfide removed as a function of the $KMnO_4$:H_2S weight ratio for a wastewater with pH = 6.8 (11). Figure 4-2 shows a plot of data collected at a Florida wastewater treatment plant where influent dissolved sulfides were 12 to 15 mg/l. $KMnO_4$ was added to the headworks for sulfide control. Dosages of 25 to 35 mg/l were generally sufficient to reduce dissolved sulfides to below 2 mg/l (12). Although this represents a sulfide removal efficiency of 80 to 90 percent, a wastewater stream containing 2 mg/l of dissolved sulfide still holds significant potential for release of gaseous H_2S to the atmosphere.

$KMnO_4$ has been used successfully for odor control in sludge handling applications, particularly dewatering, where it is added to the suction side of sludge pumps feeding the dewatering unit. Figure 4-3 shows the relationship between dosage of $KMnO_4$ and the fraction of sulfide removed (11). Total sulfide removal is achieved at dosages of 100 to 120 ppm. From a survey of 45 plants using $KMnO_4$ for sludge odor control, the average dosage was 37 ppm (13).

At one California plant, H_2S concentrations were measured above centrifuges used for dewatering anaerobically digested sludge (14). $KMnO_4$ was added immediately upstream of the centrifuges at various dosages to determine the impact of H_2S emissions. As

Figure 4-1. **Sulfide removal from wastewater using potassium permanganate (11).**

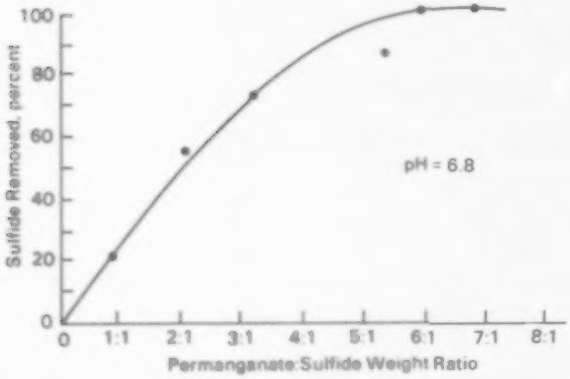


Figure 4-2. **Results of pilot studies using potassium permanganate for removal of dissolved sulfides (12).**

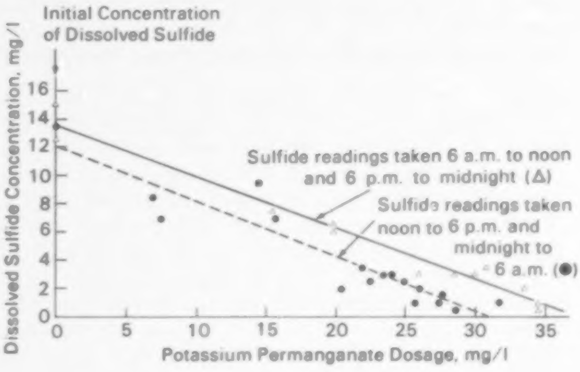
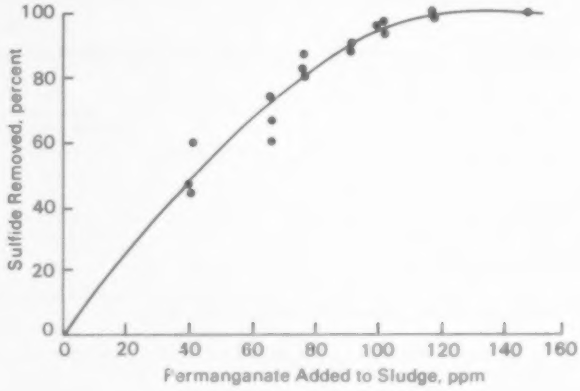


Figure 4-3. **Sulfide removal from wastewater sludge using potassium permanganate (11).**



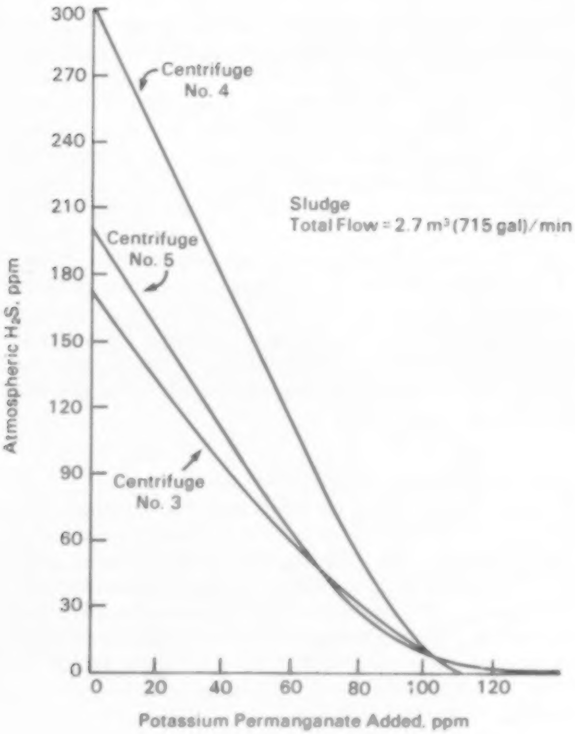
can be seen in Figure 4-4, $KMnO_4$ dosages of 110 to 120 mg/l were successful in reducing atmospheric H_2S levels to below about 5 mg/l.

c. Sodium Nitrate

Sodium nitrate ($NaNO_3$) has been successfully used for odor control in anaerobic lagoons, carbon columns, trickling filters, and sludge storage lagoons (16-20). As discussed in Chapter 3, bacteria will utilize hydrogen acceptors preferentially in the order: 1) O_2; 2) NO_3^-; and 3) $SO_4^=$. Thus, theoretically, in the absence of oxygen, no sulfide will be generated until all of the nitrate has been reduced to nitrogen gas.

Nitrate was employed for odor control at a 36-mgd industrial wastewater facility treating wastes generated from the manufacture of photographic paper, film, and chemicals (16). The plant consisted of primary settling and neutralization followed by two parallel, plastic media, super-rate trickling filters (identified as the source of the odors) which preceded a completely mixed activated sludge system. Earlier studies had indicated that the natural draft design was inadequate to ensure uniform distribution of air through the trickling filters. However, installation of forced draft ventilation improved air distribution but did not prevent odor generation. Atmospheric H_2S concentrations above the trickling filters were as high as 20 ppm, and were found to vary directly with the strength of the influent, measured as Total Oxygen Demand (TOD). H_2S emissions were most significant at influent TOD concentrations above 500 ppm. Average influent sulfate concentration was approximately 200 mg/l.

Figure 4-4. **Effectiveness of potassium permanganate addition for controlling H_2S generation from sludge centrifugation (14).**

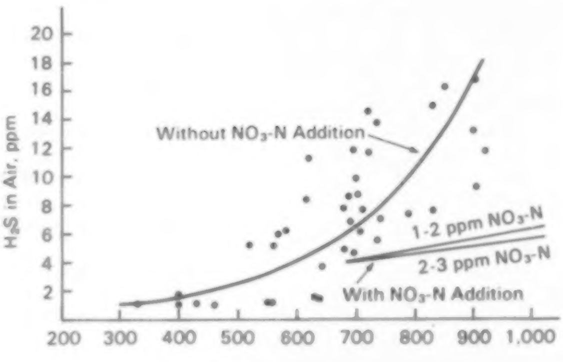


It was found that addition of approximately 5 mg/l NO_3-N to the trickling filter influent was sufficient to control most H_2S emissions from the trickling filter. Figure 4-5 shows the relationship between influent TOD and atmospheric H_2S measured above the filters, as well as the impact of nitrate addition on atmospheric H_2S emissions (16).

$NaNO_3$ was also effective for odor control in a sludge storage lagoon receiving sludge from an industrial activated sludge plant (17). The four lagoons had liquid depths of 4.5 to 6.1 m (15 to 20 ft), with surface areas ranging from 1.1 to 2.8 ha (2.7 to 7 acres). Results from the field application of a waste $NaNO_3$ solution indicated an initial dose of 20,000 mg/l NO_3-N to satisfy the initial nitrate demand and provide a nitrate residual. The waste $NaNO_3$ solution (40 percent $NaNO_3$) was distributed onto the lagoon surface using a floating boom equipped with spray nozzles. Oxidation reduction potential (ORP) was used as a measure of the potential for odor emissions. After 3 months of nitrate addition, ORP values gradually rose from -200 mV to over +200 mV and sulfide odors were eliminated (17).

Nitrate was also successfully used to control H_2S generation in both pilot-scale and full-scale tertiary

Figure 4-5. **Effect of nitrate addition on H_2S emissions from a trickling filter (16).**



activated carbon columns. Anaerobic conditions, which may develop in carbon columns due to biological growth stimulated by soluble organics in the influent, are conducive to sulfide generation. In the pilot studies, several sulfide control schemes were investigated, including routine backwashing, intermittent oxygen addition, sodium hypochlorite injection, sodium nitrate addition, and increasing the DO in the influent to the carbon columns. Chemical addition was to the influent wastewater immediately upstream of the carbon columns. Results from this investigation are summarized in Table 4-3 (18). It was found that addition of 5.4 mg/l NO_3-N, in conjunction with routine backwashing, reduced total sulfide concentrations in the carbon column effluent to zero. A greater rate of headloss development was observed, however, due to the enchanced biological activity within the carbon column during nitrate addition.

At the full-scale physical/chemical wastewater treatment system at Rocky River, Ohio, formation of H_2S in the carbon columns was the most evident day-to-day operating problem (19). Initially, copper sulfate at 6 mg/l Cu^{++} was added to the backwash water as a bactericide. Not only did this fail to control odors, it nearly doubled the turbidity in the effluent. Addition of $NaNO_3$ to the column influent at concentrations of 4 to 6 mg/l NO_3-N effectively reduced H_2S levels in the carbon column effluent from 9.5 mg/l to less than 1 mg/l during the trial (19).

Table 4-3. **Performance of Nitrate Addition Compared with Other Measures for Sulfide Control in Activated Carbon Columns (19)**

Sulfide Control Method	Total Sulfide in Carbon Column Effluent Average	Range
	mg/l	
1. Surface wash—air/water backwashing technique	2.86	1.0 to 5.7
2. No. 1 + intermittent O_2 addition to carbon column at D.O. level = 4 mg/l	1.85	1.4 to 2.5
3. Surface wash + air/water backwash + oxygenation of influent to D.O. = 2 to 6 mg/l	1.87	0.8 to 3.0
4. No. 3 + 20 mg/l Cl_2[a] to carbon influent	1.74	0 to 4.3
5. No. 3 + 40 mg/l Cl_2 to carbon influent	1.13	0.1 to 2.6
6. No. 3 + 2.9 mg/l NO_3-N[b] to carbon influent	0.30	0 to 0.95
7. No. 3 + 5.1 mg/l NO_3-N to carbon influent	0.13	0 to 0.60
8. No. 3 + 5.3 mg/l NO_3-N to carbon influent	0.05	0 to 0.26
9. No. 1 + 5.3 mg/l NO_3-N to carbon influent	0.02	0 to 0.10
10. No. 1 + 5.4 mg/l NO_3-N to carbon influent	0	0 to 0.05

[a] As sodium hypochlorite solution.
[b] As sodium nitrate solution.

d. Chlorine

Chlorine can be an effective means of odor control from wastewater unit processes. Its applications in wastewater treatment facilities have been primarily aimed at preventing odor generation from the liquid stream. Chlorine addition at the headworks is a common odor control technique for many plants. The amount of chlorine required for odor control is typically less than 80 percent of the wastewater chlorine demand. Because of the high chlorine demand of wastewater sludges, chlorine has not been used for controlling odors from sludge handling operations. Use of chlorine for sulfide control in wastewater collection systems is discussed in detail in Chapter 3.

e. Equipment, Design, and Costs

Because equipment requirements are similar to these for chemical addition to wastewater collection systems, the reader is referred to Chapter 3 for a detailed discussion of equipment and construction materials for various chemical feed systems in treatment facilities. For odor control in sludge handling operations, such as dewatering, field testing is recommended due to the variability in sludge characteristics, dewatering equipment, and conditioning agents.

Cost estimates are given in Chapter 3 for chemical addition to collection system flows of 3,785 m^3/d (1 mgd) and 37,850 m^3/d (10 mgd). These would generally be applicable to chemical addition for odor control to the headworks of a similarly sized wastewater treatment plant. Costs for chemical addition for odor control in sludge handling operations are difficult to estimate due to the wide variability in sludge characteristics, physical conditions, and plant operations. Costs will be largely dependent on the unit cost of the chemical and its dosage requirements (performance). Since performance of chemicals cannot be estimated for a "hypothetical" sludge, attempts at estimating costs are strictly subjective; thus, costs for sludge odor control by chemical addition were not developed.

4.3.1.4 Covering of Odor Producing Units

In cases where odors are generated from wastewater treatment unit processes, such as primary clarifiers, sludge thickeners, and septage holding tanks, it is often possible to construct covers or domes over the odor generating units for the purpose of containing the odors. The contained air is then passed through an air pollution control device, such as a scrubber, filter, or absorptive media, for odor removal prior to release to the atmosphere.

Domes are generally constructed of fiberglass, aluminum, or styrofoam. Inflatable domes have also been employed. Aluminum domes are available with clear spans of up to 120 m (400 ft), while fiberglass domes can be used for covering tanks having diameters up to 27 m (90 ft). Flat, low profile covers can be used for covering tanks which do not require frequent access for maintenance, such as holding tanks with little or no mechanical equipment. These may be less costly and have lower ventilation requirements, although access for cleaning or maintenance may be limited.

a. Design
Covers should be designed so as to minimize condensation problems within the dome. They must be designed to withstand wind loadings as well as static loadings resulting from snow and ice accumulation. Materials should be of sufficient thickness to prevent damage by hail. Normally, negative pressures are maintained under the domes to prevent escape of odors through openings and cracks, and to allow continuous exhausting of odorous air to subsequent treatment units.

Domes can be designed with any number of access hatches, doors, and translucent panels. Manufacturers should be contacted to determine design criteria for specific applications.

b. Costs
Several manufacturers of both aluminum and fiberglass domes were contacted during 1984 to determine a range of installed costs. Unit costs (dollars per square meter of covered surface area) varied depending on diameter, specific site conditions, and number of vents, access doors, translucent panels or other appurtenances desired. For standard domes covering tanks with diameters of 15 to 30 m (50 to 100 ft), installed costs are generally \$107 to \$160/m^2 of covered surface area (\$10 to \$15/sq ft).

4.3.2 Collection and Treatment of Odorous Air

If prevention of odor generation is not feasible or cost-effective using the various techniques previously discussed, odorous air within a confined space can be removed and treated before being released to the surrounding atmosphere. Wet scrubbers, activated carbon and other adsorptive or absorptive processes can be used to remove odor compounds from the air. Following is a discussion of these odor control processes.

4.3.2.1 Wet Scrubbers

Wet scrubbing involves contact of odorous gas with a scrubber solution, typically in a countercurrent or cross-flow fashion, to allow transfer of the odorants from the gas stream to the scrubber liquid by one or more of the following mechanisms:

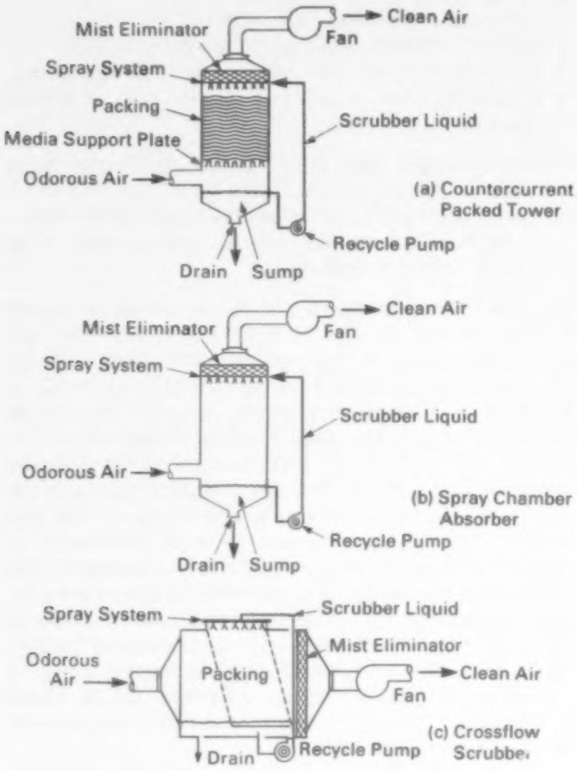
- Condensation of odorous vapors
- Removal of odorous particulates
- Odor absorption into the scrubbing solution
- Odor reaction with an oxidizing scrubbing solution
- Emulsification of odorous gases in a chemical reagent

Wet scrubbing is often ideally suited for the treatment of large air flows, greater than 1 m^3/s (2,000 cfm), contaminated with low odor threshold compounds, such as mercaptans and H_2S, at levels greater than 100 odor units per liter (20).

Scrubber designs may be of the vertically oriented, countercurrent type, or of the horizontally oriented, cross-flow type. A typical countercurrent system employs spray nozzles for injection of the scrubbing solution and an inert packing material to provide gas/liquid contact surface. The gas stream enters the bottom of the scrubber unit as evenly distributed as possible and passes through the packing material, which is irrigated with the scrubber liquid. The gas stream then passes through a mist eliminator to remove any liquid droplets, and is exhausted to the atmosphere by a fan. Wet scrubber systems are also available which generate very fine fogs or mists of scrubber liquid to achieve large surface areas for gas-liquid contact, thereby precluding the need for packing material. These are often referred to as spray chambers. Several designs of wet scrubber systems are shown in Figure 4-6.

Selection of a scrubbing liquid is dependent largely on the specific odorants to be removed. Water soluble gases such as H_2S, ammonia, and organic sulfur gases; organic nitrogen compounds such as amines; organic acids; and chlorine compounds may be removed by scrubbing with water. It is common practice, however, to use a reactive compound such as chlorine, potassium permanganate, hydrogen peroxide, or ozone in the scrubbing liquid for chemical reaction with the odorous compounds in the incoming air. In some cases, acidic or alkaline solutions can be used to neutralize the odorous compound, or to adjust the pH for better performance when used in combination with another additive. Proprietary scrubber solutions have also been developed for removal of high concentrations of specific odorous compounds such as H_2S (21). One proprietary wet scrubber system generates a dilute hypochlorite solution on-site for use as a scrubbing liquid. Wet scrubbers may employ a single pass of the scrubbing liquid with no recirculation, such as for water systems, or, more commonly, may collect and recirculate the scrubbing liquid to reduce the costs for chemical additives. Wet scrubber systems may employ single or multiple stage units depending on the nature and severity of

Figure 4-6. Typical wet scrubber systems.



(a) Countercurrent Packed Tower

(b) Spray Chamber Absorber

(c) Crossflow Scrubbe

the odor, and may utilize different scrubber solutions for each stage. Reaction times in wet scrubbers may vary from several seconds to 1 minute.

The most commonly used oxidizing scrubbing liquids are chlorine (particularly sodium hypochlorite) and potassium permanganate solutions. Hypochlorite scrubbers can be expected to remove oxidizable odorous gases when other gas concentrations are minimal. Table 4-4 indicates the expected performance of hypochlorite scrubbers for removal of odorous gases (3). Although the removal efficiencies appear high, concentrations of odorous components in the exhaust gas may still be above desirable levels, possibly requiring additional treatment. Figure 4-7 shows the dependence of effective chlorine concentration at the top of the scrubber tower on removal efficiencies of various malodorous gases (22). It should be noted that exhaust air from hypochlorite scrubbers often has medicinal, chlorine odors which may be objectionable in residential areas.

Multistage scrubber systems are often employed for odor control. Number of stages and choice of scrubber liquids depend on the characteristics and intensity of the odor, and the effectiveness of the particular chemical additives in the scrubber water. At a location experiencing H_2S odors, pilot studies were conducted using scrubbing liquids containing $KMnO_4$ and NaOH. Results are shown in Table 4-5. As a result of pilot testing with various scrubber liquids, two systems were proposed: a two-stage scrubber system using NaOH in the first stage and $KMnO_4$ in the second stage, and a one-stage scrubber using $KMnO_4$ as the scrubbing liquid (23).

Table 4-4. Effectiveness of Hypochlorite Wet Scrubbers for Removal of Several Odorous Gases (3)

Gas	Expected Removal Efficiency
	percent
H_2S	98
Ammonia	98
SO_2	95
Mercaptans	90
Other oxidizable compounds	70 to 90

a. Equipment

Equipment requirements for a wet scrubber system are dependent on the type of scrubber used and the scrubber liquid employed, volume and concentration of odorous air, and desired removal efficiency. A typical single countercurrent scrubber uses an enclosed tower containing packing material such as Raschig rings or Pall rings, a media support plenum, a scrubber solution distribution system, a mist eliminator, and an exhaust fan. Packings include plastic, ceramic, metal, or graphite with shapes ranging from rings, spheres, or saddles to porous beds of fibers (22). The scrubbing solution feed loop consists of a chemical solution tank, a metering pump and piping, a recirculation pump, and associated piping and valves. Instrumentation and controls are normally employed to monitor pressure drop through the tower and to allow addition of make-up chemicals to provide consistent quality of the scrubber liquid. Materials of construction should be corrosion resistant, although final equipment specifications will depend on characteristics of the gas to be treated and selection of a scrubbing liquid.

b. Design

Design of a wet scrubber system for odor control follows four basic steps:

1. Define the characteristics and volumes (average and peak) of the gas to be treated and the desired effluent characteristics.
2. Select a scrubbing liquid based on the chemical nature and concentration of the odorous compounds to be removed.
3. Conduct pilot tests to determine design criteria and performance.

Figure 4-7. Odorous gas removal efficiency as a function of chlorine concentration at top of packed tower (22).

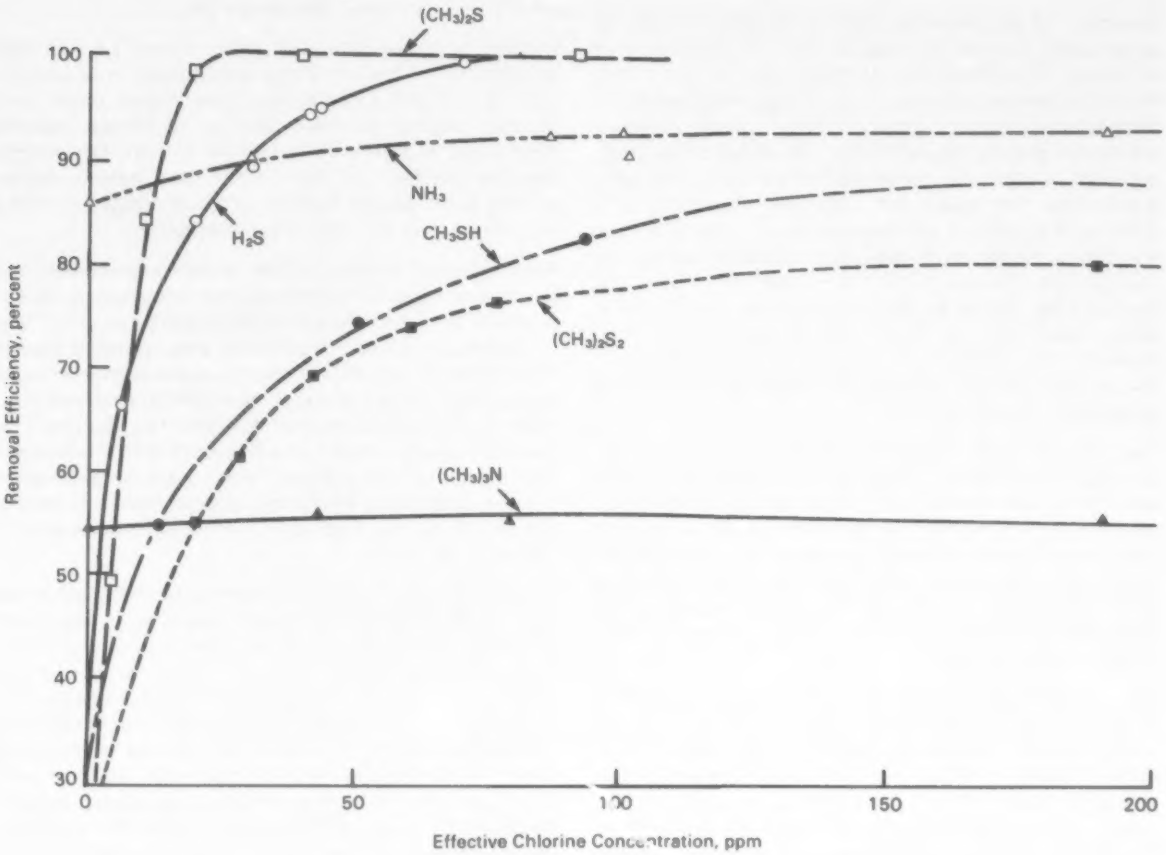


Table 4-5. Performance of Pilot-Scale Wet Scrubbers Using $KMnO_4$ and NaOH for H_2S Removal (23)

Air Flow Rate	Retention Time	Scrubbing Liquid	H_2S Concentration In	H_2S Concentration Out	H_2S Removal
m^3/min	sec		ppm		%
7.1	2.1	2% $KMnO_4$; pH = 8.6	150	18	87
3.5	4.0	2% $KMnO_4$; pH = 8.6 after 3 hrs	130	8	94
3.5	4.0	2% $KMnO_4$; pH = 8.6 after 6 hrs	130	30	77
3.5	4.0	2% $KMnO_4$; pH = 8.6 after 9 hrs	180	140	22
7.1	2.1	2% NaOH; fresh	190	2	99
12.7	1.2	2% NaOH; fresh	190	4	98
12.7	1.2	2% NaOH; after 4 hrs	90	80	12

4. Design a full-scale system based on results from pilot tests.

Full characterization of the contaminated air is an important first step in design. Diurnal fluctuations in odor intensity or concentrations of odorous compounds should be recorded continuously, if possible, in order to select peak design values. Fluctuations in air volumes should be similarly recorded. Overventilation increases total air volume and decreases intensity (concentration), which results in increase in the size and cost of the system to maintain required efficiency.

Selection of scrubber liquid(s) is critical in designing an efficient system. It is based on the chemical and physical characteristics of the contaminated air. Hypochlorite and potassium permanganate have both been used widely for control of odors from wastewater treatment plants. Hypochlorite has some advantage because it can be generated electrically on-site, precluding the need for chemical handling and storage. Potassium permanganate is effective, but scrubbers require additional maintenance to remove manganese dioxide (MnO_2), a precipitate that coats the packing. Other scrubbing solutions which have been used include water, acids, alkalies, ozone, chlorine, chlorine dioxide, and sodium bisulfite. Other agents and catalysts have been employed for removal of specific contaminants.

The ratio of liquid to gas flow is an important design consideration, since increasing the liquid-gas ratio will reduce the theoretical height of the scrubber (or the number of scrubber units). The limiting velocity of the gas is called the flooding velocity, which depends on the physical properties of the gas and the tower packing. Usually, the optimum gas velocity for contaminant removal is 50 to 70 percent of the flooding velocity (22).

The contact area between gas and liquid is important, since absorption is directly proportional to the amount of liquid surface area exposed to the gas stream. One method of increasing this area is to introduce the scrubber liquid to the tower through the use of high pressure spray nozzles which generate a fine mist or fog. Spray chambers employing high pressure nozzles can be very effective odor removal devices without the need for packing material. Another approach is to select a packing with a high specific surface area (area per unit volume). Such factors as increased pressure drop and susceptibility to clogging must be taken into account when considering such packings.

Corrosion prevention is an important consideration in design of a wet scrubber system. Usually, thermoplastics and fiber-reinforced thermoplastics are cost-effective corrosion resistant materials for small to medium size scrubbers and for most of the piping and duct work. For large units, fiberglass-reinforced plastics, stainless steel, and resin-coated mild steel are common construction materials.

Scrubber water may have to undergo treatment during use or before disposal. For example, MnO_2 particles in $KMnO_4$ scrubbers can clog nozzles and valves, and should be removed by filtration or other means if continuous operation is contemplated. If operation is intermittent, accumulated sludge should be drained periodically. Scrubber water disposal is generally not a problem at a wastewater treatment plant, since the spent scrubber water can be introduced back into the wastewater stream at a rate which will not upset the processes.

Design of a wet scrubber system may be complex depending on the intended application, characteristics of the gas stream, scrubber liquid used, and design objectives. Pilot testing is almost always recommended prior to full-scale design. Equipment representatives can be of significant help in determining such design factors as tower height, packing materials, and scrubber liquid selection.

An example of a wet scrubber system application for a municipal treatment plant sludge processing operation is at the 378,500 m^3/d (100 mgd) Southerly Plant in Cleveland, Ohio. This facility also handles sludge from the 568,000 m^3/d (150 mgd) Easterly Plant and uses low pressure oxidation (LPO) and vacuum filtration to condition and dewater the sludge. The oxidized sludge from the LPO system is discharged into a wet well, where dissolved gases are allowed to escape from solution. Air exhausted from this well is burned in a dual fuel (gas/oil) fume incinerator at 760°C (1,400°F).

Air from the four LPO decant tanks is exhausted to the odor control system, which is a two-stage packed bed scrubber. Chlorine gas is injected directly into the first-stage scrubber. This scrubber provides the contact media and contact time for the chlorine and odorous air. The second-stage scrubber uses water to remove any heavy chlorine odor. It was determined that the compound causing the odor was an acid aldehyde and that the chlorine scrubber system removed 39 percent of this substance. Air from the vacuum filter room and vacuum pump exhaust is also treated with a wet scrubber, this one using a potassium permanganate scrubbing solution.

c. Costs

Typical costs for wet scrubber systems for odor control are presented in Table 4-6. Note that an inlet H_2S concentration of 20 ppm has been assumed. Although H_2S concentrations well in excess of this value have been observed, H_2S concentrations can be expected to fluctuate widely, depending on the size of the building, types of equipment housed, characteristics of wastewater or sludge being processed, time of day, and scheduling of treatment operations. For purposes of this analysis, an outlet concentration of <1 ppm has been assumed, representing a required removal efficiency of 95 percent. In actual practice, lower outlet concentrations may be required, since 1 ppm is well above the odor threshold.

These costs are budget level estimates and are accurate to within +30 percent, -20 percent for a

Table 4-6. Typical Costs for Wet Scrubbers for Odor Control (1984 $)[a]

Design Air Flow	Capital Costs[b]	Annual Chemical Costs
m^3/min	$	$/yr
28	39,000	2,000
280	77,000	19,500

[a] Assumed conditions:
1. Continuous operation
2. Inlet H_2S = 20 ppm
3. Outlet H_2S = < 1 ppm
4. Scrubber liquid = NaOCl + NaOH solution
5. Chemical Costs:
   NaOCl: $1.41/kg
   NaOH: $0.44/kg

[b] Including scrubber tower, packing, recirculation pump, fan and ductwork, appurtenances and controls, installation.

typical case. In practice, detailed estimates of capital, operating, and total present worth costs should be developed when comparing costs of alternative odor control technology for a specific application. In addition, expected performance and reliability should be assessed and ranked for each of the various alternatives investigated in order to select the most cost-effective and reliable system that will meet the desired objectives.

4.3.2.2 Activated Carbon Adsorption

Activated carbon adsorption is a commonly used method for treatment of malodorous air. It has been used in wastewater treatment plants as a primary odor control system and as a polishing step following other alternatives such as scrubbers. Adsorption is the phenomenon whereby molecules adhere to a surface with which they come in contact. Activated carbon has a high surface-to-volume ratio; thus, a large surface area is available for adsorption in a relatively small volume. The physical characteristics of activated carbon are shown in Table 4-7.

Due to the non-polar character of the surface, activated carbon adsorbs organic, and some inorganic compounds, in preference to water vapor. The quantity of materials adsorbed is partially dependent on the physical and chemical characteristics of the compound. In general, organic compounds with molecular weights greater than 45 and boiling points over 0°C will be readily adsorbed. Adsorption of organic compounds is relatively nonselective; that is, it is not strongly affected by solubility or chemical class of the compounds. Under normal conditions, adsorptive capacity of activated carbon can reach 5 to 40 percent of the weight of the activated carbon (24).

Table 4-7. Physical Characteristics of Activated Carbon for Odor Control

Parameter	Value
Suface area, m^2/g	950
Surface area, m^2/cm^3	380 to 600
Pore volume, cm^3/g	0.6 to 1.0
Pore volume, cm^3/cm^3	0.24 to 0.50
Mean pore diameter, angstroms[*]	15 to 20

[*] Refers to micropore volume (<25 angstrom diameter); Macropores (>25 angstrom) not included.

The quantity of material that can be adsorbed in a bed of activated carbon depends on the following factors:

1. Concentration of material in the space around the activated carbon
2. Total surface area of the activated carbon
3. Total pore volume
4. Temperature
5. Presence of other competing contaminants
6. Characteristics of the compounds to be adsorbed (molecular weight, boiling point, polarity, size, shape)
7. Polarity of the activated carbon
8. Relative humidity of vapor stream
9. Contact time of vapor stream within the activated carbon bed

Maximum adsorbing capacity is favored by a high concentration of the substance surrounding the activated carbon, large surface areas, freedom from competing contaminants, low temperature, and aggregation of the contaminant in large molecules that fit and are strongly attached to the receiving sites on the adsorbent (22).

The nonselectivity of activated carbon has an advantage due to the ability to remove complex mixtures of odorous compounds. However, nonselectivity can present a disadvantage in that the capacity of the carbon can be exhausted prematurely by the adsorption of nonodorous hydrocarbons. A pilot study conducted in Sacramento, California attempted to investigate the possible occurrence of this phenomenon in an evaluation of activated carbon adsorption for treatment of sewer off-gases (25). It was found that the useful life of the carbon prior to odor breakthrough was 1.5×10^5 to 4×10^5 air volumes per carbon volume, or 276 to 735 m^3/kg (4,420-11,800 ft^3/lb) of activated carbon. Data indicated greater than 90 percent removal of hydrogen sulfides and total hydrocarbons prior to odor breakthrough. After odor breakthrough, the activated carbon was still removing up to 37 percent of the H_2S and 71 percent of the hydrocarbons. These results are shown in Table 4-8.

Based on data collected, carbon bed life was estimated for hydrocarbon saturation and H_2S saturation. It was found that the useful life based on hydrocarbon saturation, 75 m^3/kg (1,200 ft^3/lb), was two orders of

Table 4-8. Pilot Study on Sewer Odor Control Using Activated Carbon at Sacramento, CA (25)

	Day 1			Day 2*			Day 3*			Day 4*		
Parameter	In	Out	% Rem.	In	Out	% Rem.	In	Out	% Rem.	In	Out	% Rem.
CH_4, ppm	265	270	0	460	440	4	165	161	2	41	50	0
CO_2, percent	0.4	0.3	0	0.6	0.7	0	0.2	0.2	0	0.1	0.1	0
H_2S, ppm	1.1	0.1	91	11.0	7.0	36	15	9.4	37	<0.1	<0.1	0
Total hydrocarbons, ppm	59.1	2.8	95	531.4	117.0	67	76.1	52.1	32	12.9	3.8	71
Odor conc., odor units	15	<2		16,700	18,800		75	75		66	75	

*Sample taken after odor breakthrough.

magnitude lower than for H_2S saturation, 8,800 m^3/kg (141,000 ft^3/lb). However, useful life based on odor breakthrough governs practical design of an activated carbon system (25).

An interesting observation in this and other pilot studies was that H_2S odor in sewer off-gas is altered by antagonistic or inhibiting effects of other substances in the exhaust gas such that there is little direct relationship between the actual H_2S concentration and the odor concentration as measured by an olfactometer. Removal of antagonistic or inhibitory hydrocarbons by the activated carbon can result in significantly higher perceived odor concentrations from a given chemical concentration of H_2S. This has been observed on activated carbon units treating off-gases from headworks and primary clarifiers, where the gas odor characteristics can change through the carbon column. Thus, odor breakthrough can occur while H_2S and hydrocarbons are still being removed due to reductions in antagonistic or inhibitory hydrocarbons that can modify the odors. The practical result is that useful carbon life based on odor breakthrough may be significantly less than if based on hydrocarbon saturation (25).

A special activated carbon impregnated with caustic (NaOH or KOH) is often specified for odor control applications in wastewater collection and treatment works. H_2S is adsorbed on the carbon surface, and reacts to form elemental sulfur and sulfates.

a. Equipment

Thin-bed (about 2-cm) carbon adsorbers can provide a useful service life if odor concentrations are low (< 5 ppm) and the effective mass transfer zone for adsorption is very short (rapid adsorption kinetics). Thin-bed adsorbers have an advantage of low resistance to air flow. Activated carbon is retained between perforated metal plates in flat, cylindrical or pleated shapes. Cylindrical canisters are commercially available to handle about 0.7 m^3/min (25 cfm), while larger pleated cells can handle 21 to 28 m^3/min (750 to 1,000 cfm), and systems comprising aggregates of flat-bed components can handle 57 m^3/min (2,000 cfm) (22).

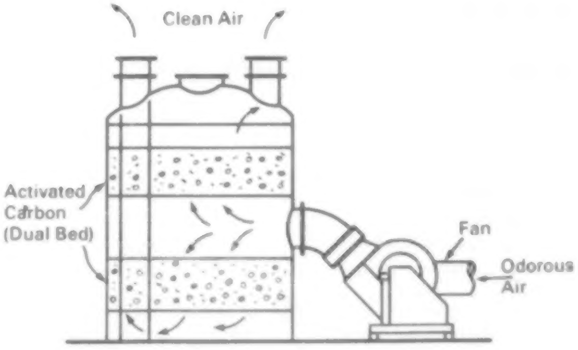
Deep-bed adsorbers are generally used where odor concentrations > 5 ppm are present, or when on-site carbon regeneration is used. Bed depths range from 0.3 to 1.8 m (1 to 6 ft). Design capacities range up to 1,130 m^3/min (40,000 cfm). Experience has shown that a 0.9-m (3-ft) deep carbon bed offers sufficient depth to provide reliable treatment efficiency without causing excessive pressure drop.

Relatively little mechanical equipment is required with an activated carbon odor control system other than the vessel containing the adsorbent. In some cases, contaminated air may be pretreated by using a grease filter and a condensing unit, which results in temperature reduction of the air stream and more efficient operation. A typical activated carbon adsorber is shown in Figure 4-8.

A typical activated carbon odor control facility would consist of air pretreatment units (optional), activated carbon adsorber unit(s), exhaust fan(s), and associated piping and ductwork. Back-up carbon storage is not normally required.

Activated carbon is typically regenerated thermally, although special activated carbon, designed for odor control applications and impregnated with caustic

Figure 4-8. Typical activated carbon filter for odor control.



(NaOH or KOH), can be regenerated chemically using a solution of NaOH or KOH. Thermal regeneration using multiple hearth furnaces requires removal of the carbon from the adsorption system. However, chemical regeneration of NaOH- or KOH-impregnated carbon is conducted in situ using a 50-percent NaOH solution. Due to its lower cost per unit weight and its ability to dissolve more sulfur per unit weight than KOH impregnate, NaOH-impregnated carbon and NaOH solution use for regeneration can result in significant savings in operation and maintenance costs.

b. Design

The design of activated carbon adsorbers involves four fundamental steps:

1. Characterize the contaminated air (volumes, concentrations of constituents) and the desired effluent characteristics.
2. Select the adsorbent.
3. Conduct pilot studies to determine expected performance, useful life of carbon, design criteria, etc.
4. Apply pilot data to full-scale design.

The importance of full characterization of the contaminated air cannot be overemphasized. Of particular importance is the diurnal variation in air volumes and odor intensity. Failure to accurately estimate these parameters may result in poor performance or higher than anticipated carbon replacement or regeneration frequency. Characterization of specific constituents may also be important in design, since special impregnated activated carbons now available may demonstrate improved performance over standard carbons. For example, activated carbon impregnated with sodium hydroxide is often recommended for use when sulfur-based compounds, such as H_2S and mercaptans, are the principal odor-causing materials. In addition to claims of its superior performance, the carbon can be regenerated in situ using a commercial grade caustic in 50-percent solution.

Since exhaust air from sludge handling buildings, wet wells, and covered process tankage often contains a complex mixture of odorous compounds, pilot testing is essential prior to full-scale design. Objectives in pilot testing include defining expected performance (removal efficiency), estimating the useful life of the carbon, determining effectiveness and ease of carbon regeneration, and developing design criteria for the full-scale system. In addition to analytical determination of specific compounds such as H_2S and methyl mercaptan, an olfactometer should be used as a measure of total odor removal through the system. Odor breakthrough curves should be developed to assist in determining frequency of carbon regeneration or replacement.

An initial approximation of carbon life before odor breakthrough can be gained by using Turk's equation, (3):

$$t = \frac{(W)(S)}{(E)(R)(C)} \tag{4-4}$$

where,

t = carbon life, days

W = mass of adsorbent, g

S = proportion of maximum adsorption of adsorbent, 0.16 to 0.5

E = average efficiency of carbon adsorber

R = gas flow rate, m^3/d

C = concentration of odorant, g/m^3

c. Costs

Table 4-9 contains estimated costs for activated carbon adsorbers for odor control at air flow rates of 28 m^3/min (1,000 cfm) and 280 m^3/min (10,000 cfm). It is assumed that under continuous operation, H_2S concentration at the inlet to the adsorber would be 20 ppm average, and at the outlet, less than 1 ppm. In practice, use of wet scrubbers may be recommended ahead of the carbon adsorbers for this inlet H_2S concentration. Costs are also provided for carbon adsorbers for an inlet H_2S concentration of 10 ppm.

A specially manufactured activated carbon impregnated with NaOH is normally specified for H_2S removal applications. This carbon can be regenerated within the vessel using a 50-percent NaOH solution.

Table 4-9. Typical Costs for Activated Carbon Adsorbers for Odor Control[a] (1984 $)

Design Air Flow	Capital Costs[b]	Annual Costs[c]
m^3/min	$	$/yr
28	29,800	6,200
280	128,000	48,000

[a] Assumed conditions:
1. Continuous operation
2. Inlet H_2S = 20 ppm
3. Outlet H_2S = <1 ppm
4. Caustic impregnated carbon
5. Carbon costs:
   a. 28 m^3min system: $6.67/kg ($3.03/lb)
   b. 280 m^3min system: $5.48/kg ($2.49/lb)
6. Regeneration chemical (NaOH) cost = $0.17/L ($0.65/gal)

[b] Including adsorber vessel, fan, ductwork, appurtenances and controls, installation.

[c] Including initial and replacement carbon plus regeneration chemicals.

NOTE: Costs are for activated carbon adsorption alone; in practice, use of wet scrubbers ahead of carbon may be recommended for inlet H_2S concentrations of 20 ppm.

The 28-m^3/d (1,000-cfm) system would consist of a single-vessel single-bed unit, while the 280-m^3/d system would employ two dual bed units.

The costs shown are budget level estimates (+30 percent, -15 percent), presented to show typical costs of activated carbon adsorbers for odor control. In practice, detailed cost estimates would be required for site-specific analysis of alternative odor control technologies.

4.3.2.3 Other Adsorption Processes

Other adsorptive media besides activated carbon have been used for odor control applications. Two such media are activated alumina impregnated with potassium permanganate and wood chips mixed with iron oxide. These alternatives are primarily suited for small installations with relatively low volumes of malodorous air requiring treatment.

Several commercial products are available which consist of dry pellets of activated alumina impregnated with potassium permanganate. Odorous compounds are adsorbed into the surface of the pellets and are subsequently oxidized by the potassium permanganate. Pellet diameters are typically 3 to 9 mm (⅛ to ⅜ in), and they contain approximately 5-percent $KMnO_4$ by weight. Contaminated air is passed through a deep bed or series of shallow beds containing the media, and exhausted to the atmosphere. Prefabricated package systems are available to handle up to 280 m^3/min (10,000 cfm) of contaminated air.

Packaged odor control systems using $KMnO_4$-impregnated alumina are typically horizontal or vertical flow units employing a prefilter and/or a mist eliminator for removal of particulates and moisture, a series of 7.6-mm (3-in) deep beds containing the media, and a blower. For some applications, a final filter may be used.

The potassium-permanganate-impregnated activated alumina pellets have a finite capacity for removal of odorous compounds. When the capacity is exhausted, it is discarded, as this media cannot be regenerated. The useful lifetime of the media is dependent on the total mass throughput of the odorous contaminant. For H_2S, it has been estimated that 1 kg of media will remove 0.076 kg of H_2S gas before it is exhausted. Typical performance of a 3.2-cm (1.25-in) deep bed of $KMnO_4$-impregnated alumina for removal of H_2S is shown in Figure 4-9 (26). Such a system would not be practical for treating air contaminated with high concentrations of H_2S, and is more suited for relatively clean air applications such as control rooms, computer rooms, etc.

The use of iron oxide filters for odor control has been investigated at several locations, including the U.S. and Norway. Such systems typically incorporate a bed of wood chips mixed with iron oxide (Fe_2O_3). Contaminated air is passed upward through the bed, where odorous contaminants are adsorbed by the media. The postulated mechanism for H_2S removal is described by the following reaction:

$$Fe_2O_3 + 3\,H_2S \longrightarrow Fe_2S_3 + 3\,H_2O \qquad (4\text{-}5)$$

A schematic diagram of an iron oxide filter is shown in Figure 4-10.

Figure 4-9. **Performance of $KMnO_4$-impregnated activated alumina adsorbers on hydrogen sulfide (26).**

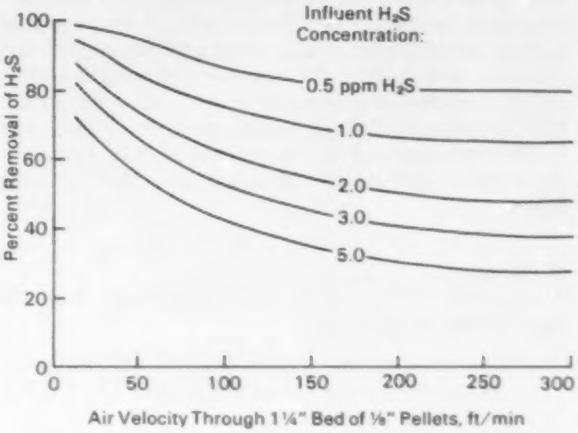
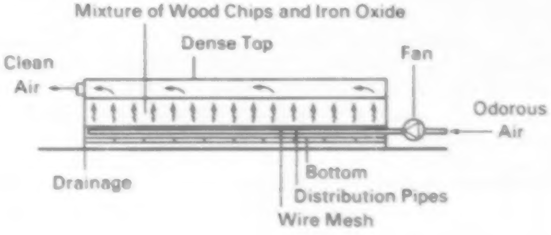
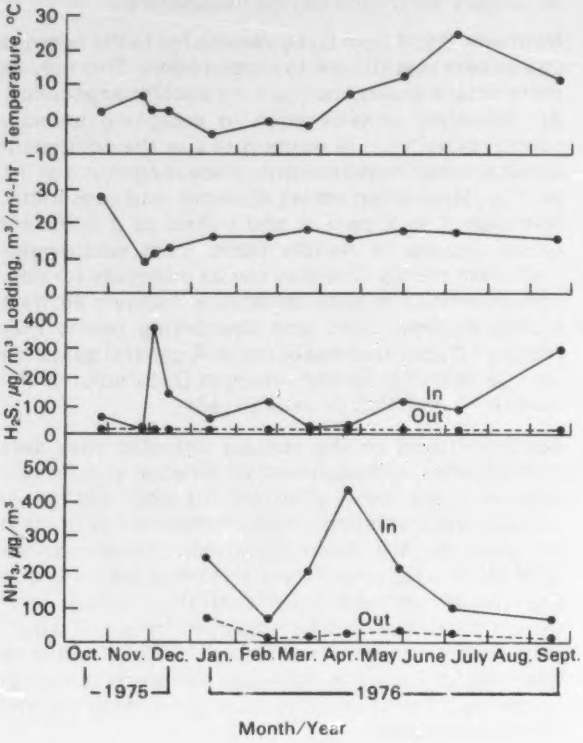


Figure 4-10. **Typical iron oxide filter for odor control.**



A pilot system was studied in Norway, where wood chips were mixed with iron oxide particles in the proportion of 0.2 kg Fe_2O_3/kg wood chips. The bed was 0.4-m (1.3-ft) deep and was loaded at a rate of 16 to 18 m^3/m^2/hr (0.9 to 1.0 cfm/ft^2). Figure 4-11 shows the results of the study. H_2S concentrations in the incoming air ranged from 10 to 380 μg/m^3, while concentrations in the exhaust air ranged from 0 to 3.3 μg/m^3. H_2S removal efficiencies were from 90 to 100 percent. Ammonia removal was somewhat less, ranging from 70 to 100 percent (27).

Figure 4-11. Performance of an iron oxide filter for odor control (27).



4.3.2.4 Ozone Contactors

Ozone is a powerful oxidant that has been used for odor control, particularly in industrial applications. The principle of ozone treatment of odorous gases is that, given sufficient time for contact of the odorous compounds with ozone, the odors will be eliminated by chemical oxidation. Ozone can be utilized for the oxidation of numerous odor-causing compounds. Several examples of ozone reactions in odor control applications are given below (28).

Hydrogen sulfide:

$$H_2S + O_3 \xrightarrow{\text{major path}} S + H_2O + O_2 \qquad \text{(4-6)}$$
$$\xrightarrow{\text{minor path}} SO_2 + H_2O$$

Amine:

$$R_3N + O_3 \longrightarrow \underset{\text{(amine oxide)}}{R_3N\text{-}O + O_2} \qquad \text{(4-7)}$$

Methyl mercaptan:

$$CH_3SH + O_3 \longrightarrow CH_3\text{-}S\text{-}S\text{-}CH_3 + O_3 \longrightarrow \underset{\text{(methyl sulfonic acid)}}{CH_3\text{-}SO_3H + O_2} \qquad \text{(4-8)}$$

Ozone is a very unstable gas which requires generation on-site. For applications requiring greater than 0.9 kg (2 lb)/d of ozone, ozone is generated using the corona discharge principle (29). This involves passage of a parent gas such as oxygen or pre-treated air through a discharge gap, across which a high voltage is applied. The resulting gas contains ozone at relatively low concentrations of 1 to 2 percent by weight. Package ozonators are available which employ the corona discharge principle.

For economical operation of an ozone generator, heat must be efficiently removed from the system, and the feed gas must be clean, cool, and dry. Ozone decomposes more readily at high temperatures, thus requiring cooling systems using air or water. Water and impurities in the parent gas can cause generation of fouling agents which may coat the dielectrics, thus lowering ozone production and increasing power consumption. Moisture in the feed gas may also cause formation of nitric acid in the corona, which is corrosive and may result in reduced efficiency and increased maintenance.

Ambient air processing equipment includes compressors, heat exchangers, and various size filter units. A typical air filtration system may include an air conditioning filter, a 50-micron filter for the compressor intake, a 5-micron filter, and a 4-angstrom molecular sieve. The molecular sieve is used for the removal of water vapor, carbon monoxide, carbon dioxide, nitrogen, methane, and H_2S. Conditioned air is delivered to the ozone generator at a dry −51 °C (−60°F) frost point. Alternatively, pure oxygen can be used as the parent gas, precluding the need for pretreatment (29).

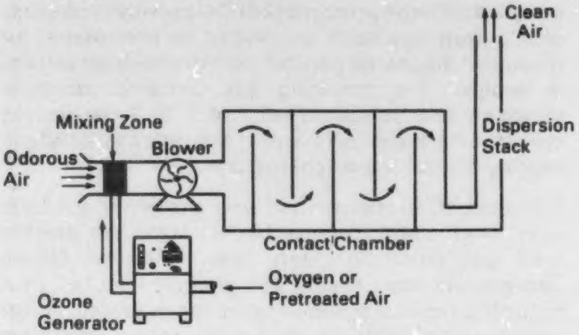
The resulting gas stream, containing 1 to 2 percent ozone by weight, is then introduced into a baffled contact chamber to allow mixing and contact with the air to be deodorized. Movement of the odorous air through the contact chamber is effected through the use of fans. Contact times may vary from 3 to 60 seconds, although 15 seconds is generally recommended as a minimum. A typical ozone generator and contact chamber system is shown in Figure 4-12 (29).

a. Equipment

For many odor control applications, a package ozone generator may be purchased. Most applications requiring less than 45 kg (100 lb)/d of ozone employ ambient air as the parent gas, while larger systems may use pure oxygen, either supplied in containers as a liquid, or generated on-site. For odor control applications use of oxygen as a feed gas is more likely to be economically impractical, since it cannot be recycled as it can for other applications, such as disinfection of treatment plant effluents.

Equipment requirements for an air-fed ozone system consist of an air pretreatment system (compressors,

Figure 4-12. Typical ozone system for odor control (29).



heat exchanger, air filters, molecular sieve); an ozone generator and diffuser or injector; a baffled contact chamber; and appropriate piping, valves, appurtenances and controls. Ozonators generally come equipped with a number of safety features, including a pressure switch on the air circuit, a safety relief valve and flow switch on the cooling water circuit, microswitches on cabinet doors to automatically shut off power when the doors are opened, and audible and visible alarms. Controls are electrically interlocked for shutdown in the event of malfunction (30). Ozone dose may be controlled by a manual setting, or by the use of an ozone residual meter, which by monitoring ozone concentrations in the treated exhaust air sends a signal back to the generation equipment that increases or decreases the amount of ozone fed to the contact chamber.

Ozone is a powerful oxidizing agent, second only to fluorine, and thus some care must be exercised in selecting materials. Stainless steel is preferred for piping materials in contact with ozone, although unplasticized PVC has been used. Ozone will attack the unsaturated portions of the plasticizer molecule, and eventually soften normal PVC. Epoxy resin is not suitable for joining PVC pipe exposed to ozone, requiring either threading and use of Teflon tape, or hot welding. Teflon tape is inadequate for sealing stainless steel joints; Teflon tape with RTV cement has been found to be satisfactory. Gaskets should be manufactured from fluoroelastomers, as should the diaphragms in an ozone metering pump (31). Ozone contact chambers have been constructed of concrete, fiberglass, PVC, transite, and stainless steel.

b. Design

Design criteria for an ozone odor control system include: 1) type of odor; 2) concentration of the odor; 3) temperature of the exhaust gases; 4) humidity of exhaust gases; 5) retention time within the contact chamber; and 6) the distribution of ozone within the contact chamber (32).

In many cases, the feasibility of ozone oxidation for odor control can be established in the laboratory. However, on-site pilot testing is often necessary to accurately determine design parameters.

Normally, 3 to 4 ppm O_3 by volume fed to the odorous gas stream is sufficient to control odors. This may be more or less depending upon the specific application. An important consideration in designing an odor control system using ozone is to size the equipment based on peak requirements, since magnitude of the odor problem often varies diurnally and seasonally. Although 1 to 2 ppm is often cited as a sufficient ozone dosage to handle odors from wastewater treatment plants, this may not be adequate for odor concentrations at peak conditions. Exhaust air from sludge storage tanks and dewatering rooms may require 10 ppm of ozone or more. A general guideline for H_2S oxidation is that one ppm O_3 by volume will oxidize 10 ppm H_2S by volume (33).

Reaction times in the contact chamber may vary considerably. Although contact times of as little as 7 seconds have been effective for odor control at wastewater treatment plants, contact times of 30 to 40 seconds are more commonly recommended (30)(34). The required detention time is dependent on the type of odor and its concentration, and general design criteria cannot be employed for specific odor control applications. The contact chamber should be designed for complete, intimate mixing and contact of ozone with the odorous air to ensure complete and efficient oxidation.

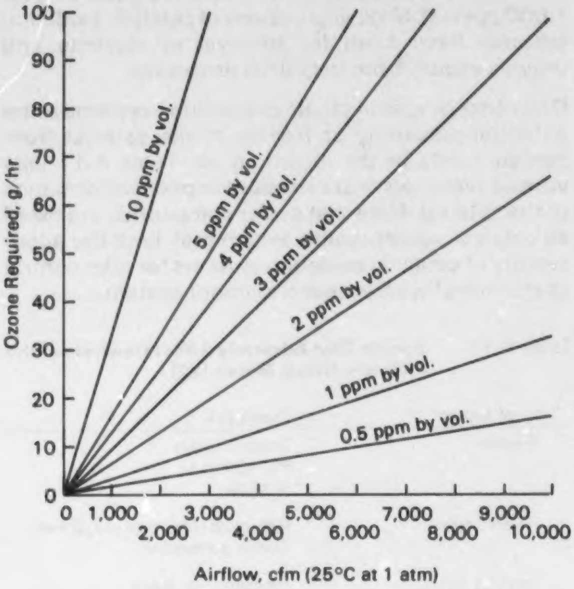
Given the necessary dosage of ozone for oxidation of odorous compounds and the air flow rate, Figure 4-13 can be used to estimate the ozone requirement in grams per hour (28). Thus, for an air flow of 140 m^3/min (5,000 cfm) and a required dosage of 3 ppm, the ozone requirement would be approximately 50 grams per hour, or 1.2 kg (2.6 lb)/d.

Other important design considerations when considering odor control by ozonation are the occupational and environmental health and safety aspects. OSHA has set a maximum 8-hour continuous occupational exposure level of 0.1 ppm for ozone. Ozone can be detected by the human nose at concentrations as low as 0.04 ppm, which often allows detection of leaks before ozone concentrations become hazardous. Ozone dosage should be adjusted to minimize discharge of unreacted ozone. Ozone monitors can be installed at the discharge of the contact chamber to either trigger an alarm or send an electrical signal to the ozone generator to increase or decrease the rate of generation.

c. Costs

Typical costs for ozone systems are shown in Table 4-10 for air flow rates of 28 m^3/min (1,000 cfm) and

Figure 4-13. Ozone requirements for various air flows and ozone dosages (28).



280 m^3/min (10,000 cfm). The costs shown in Table 4-10 do not include the cost of the ozone contactor, which is normally furnished as fiberglass-reinforced plastic (FRP) baffled chamber for installations requiring less than 4.5 kg (10 lb)/d. For larger installations, existing tankage or buildings may be converted into the necessary contact chamber. The contact chamber volume required for an air flow of 28 m^3/d would be approximately 4 m^3 (150 cu ft) for an influent H_2S concentration of 20 ppm. The cost of the contact chamber can be conservative, estimated at 8 to 12 percent of ozone capital cost shown in Table 4-10.

Table 4-10. Typical Costs for Ozone Systems for Odor Control[a] (1984 $)

Air Flow	Capital Costs[b]
m^3/min	$
28	30,400
280	67,500

[a] Assumed H_2S inlet concentration = 20 ppm; outlet concentration <1 ppm.

[b] Includes skid mounted ozone generator with liquid ring compressor, electric reactivated dual tower, automatic air dryer, transformer/reactor, controls, air filter and silencer, air compressor, precooler, prefilter, dessicant air dryer, after filter, after cooler and necessary pressure, temperature and air flow regulators, indicators and valves, and electrical control panel. System does not include cost of ozone contactor.

4.3.2.5 Combustion Processes

Combustion can be an effective means of odor control, since it offers nearly total oxidation of the odorous compounds. Combustion may be employed when malodorous gases are toxic, have a low concentration of combustibles, and have no heat recovery value. Hydrocarbons are oxidized to CO_2 and water, and nitrogen and sulfur-bearing compounds are oxidized to oxides of nitrogen and sulfur, respectively. It is important, however, that combustion be complete; otherwise intermediate products may be formed that are malodorous.

Two types of combustion processes are used in odor control applications: direct-flame oxidation and catalytic oxidation.

In the direct flame process, odorous gas is mixed with combustion air and exposed to temperatures of 480 to 815°C (900 to 1,500°F) and velocities of 4.6 to 9.1 m (15 to 30 ft)/s, with combustion chamber residence times of 0.25 to 0.60 second giving satisfactory results (20). Longer residence times have been employed. Incoming contaminated air should have a heating value of at least 3,700 kJ/m^3 (100 Btu/ft^3). Heating value can be estimated by the following (35):

$$\text{Gross Btu/lb} = (145.4)(C) + (620)(H) - (0.125)(O_2) + (41)(S) \qquad (4\text{-}9)$$

where,

C, H, O_2, S = weight percentages of carbon, hydrogen, oxygen, and sulfur, respectively, in the air.

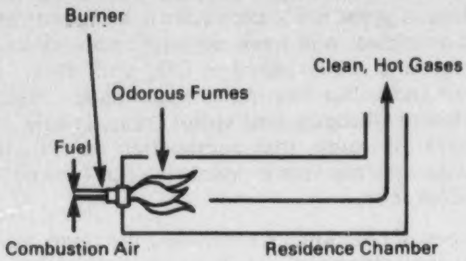
Small amounts of fuel may be required if the odorous fumes are below the lower flammable limit. The concentration in the fume-air mixture must lie between the lower and upper flammable limits.

A conventional direct flame oxidation system is shown schematically in Figure 4-14 (20). Also shown is a direct flame system incorporating heat recovery and preheating of the contaminated air.

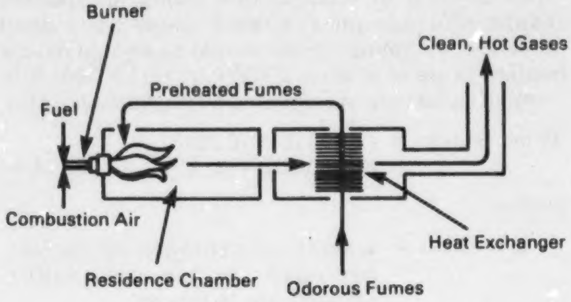
In the catalytic oxidation process, the presence of a catalyst allows oxidation to take place at a lower temperature and in the absence of a flame (22). Although the reaction does not take place in the presence of a flame, a burner preheats the odorous air to raise the temperature required for catalytic oxidation. In general, the temperature of the air upstream from the catalyst is 315° to 480°C (600° to 900°F), and 425° to 590°C (800° to 1,100°F) downstream of the catalyst. A schematic diagram of a catalytic oxidation system is shown in Figure 4-15 (22).

The catalyst is usually finely divided platinum or palladium on a ceramic supporting structure. However, other metals such as nickel, copper, chromium,

Figure 4-14. Direct flame oxidation systems (20).

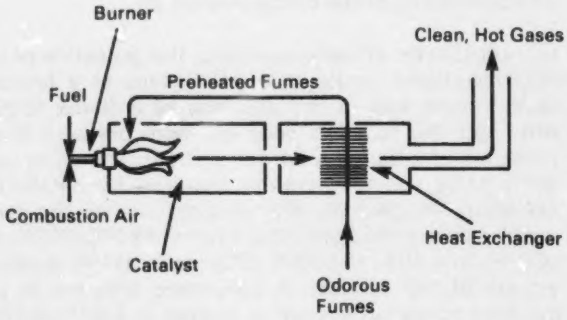


a) Conventional Direct Flame Combustion



b) Direct Flame Combustion with Heat Recovery

Figure 4-15. Catalytic oxidizer with heat recovery (22).



and manganese, are used. Although the precise mechanism of catalytic combustion is not well understood, it proceeds through three steps: 1) adsorption on the active surface; 2) chemical reaction (oxidation) on the surface; and 3) desorption of the reaction products. One major limitation of catalytic oxidation is that it is generally recommended for use only when the concentration of odorous gas is at least 1,000 ppm (3). Most applications of catalytic oxidation systems have been for removal of solvents and organic vapors from industrial processes.

One concern with catalytic combustion systems is the potential poisoning or fouling of the catalyst from contaminants in the incoming air. Table 4-11 lists various materials that can result in poor performance of the catalyst. Note that sulfur compounds are listed as catalyst suppressants, which may limit the applicability of catalytic oxidation systems for odor control in municipal wastewater treatment systems.

Table 4-11. Agents That Adversely Affect Catalysts of the Platinum-Group Metals (22)

Type of Agent	Examples
Poison	Heavy metals Phosphates Arsenic
Suppressant	Halogens (free and combined) Sulfur compounds
Fouling agent	Inorganic particles Alumina and silica dust Iron oxides Silicones

Advantages of direct flame oxidation over catalytic combustion include lower maintenance costs, less downtime, and better odor control. A catalytic system may require less fuel but may have higher overall power requirements (3).

Choice of one system over another will depend on the specific application and the characteristics of the gas to be treated.

a. Equipment

There are many commercially available combustion devices marketed for odor control applications. Each design is somewhat different, and manufacturers should be contacted to determine specific details regarding design and construction of particular units.

Little additional equipment is required for a direct-flame or catalytic combustion system. Ductwork is required to convey the contaminated air to the unit, and high temperature exhausts must be discharged through a stack to prevent human contact. A suitable source of fuel must be available. Temperature sensors should be located in the influent air ductwork, in the combustion chamber, and in the exhaust stack to ensure proper temperatures are maintained for complete combustion. Heat exchangers are often used to defray fuel costs for preheating influent air.

Materials of construction include high temperature-resistant metals, insulating materials, and refractory materials that are used in the combustion chamber to withstand temperatures of 650° to 760°C (1,200° to 1,400°F).

b. Design

Major considerations in design of thermal combustion systems for odor control include residence time, temperature, and mixing. Although mixing is often overlooked, it may be one of the most important parameters in design of an efficient system. Ineffective operation can often be traced to poor mixing characteristics (35).

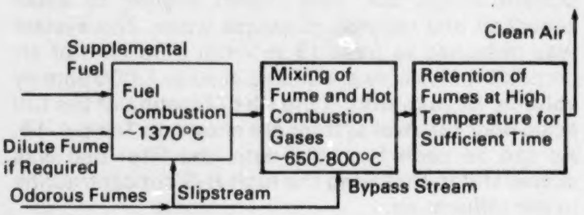
Temperature required for complete combustion is dependent on the compounds to be oxidized. Generally, in direct flame units, temperatures of 480° to 815°C (900° to 1,500°F) must be maintained. Relatively small temperature drops can result in substantial loss in oxidation efficiencies of specific compounds.

Residence time is closely interrelated with temperature. Increased temperature results in expansion of the gas and therefore lowers residence times in the combustion chamber. Typical residence times required for odor destruction range from 0.25 to 0.6 second.

Figure 4-16 shows a flow diagram for a direct flame combustion system. As shown, a portion of the contaminated air is bypassed directly to the combustion chamber, the remainder being mixed with air. This proportion is generally about 50 percent. The mixture of air and slipstream enters the combustion chamber, and is then mixed with the bypass contaminated air. Achieving good mixing of these two streams is critical for efficient operation. The mixture must be retained for a sufficient time to allow complete oxidation of the malodorous components.

Proper mixing can be accomplished through proper selection and sizing of burners, combustion chambers, and reactor configurations. One approach is the use of burners distributed in the combustion chamber area (35). Location and size of burners is important. They must be placed where the flow is fully turbulent and where good mixing conditions prevail. Velocities of 3 to 9 m/s (10 to 30 ft/s) are generally sufficient for molecular mixing (35). Baffles have also been effective for improved mixing.

Figure 4-16. Flow diagram for a direct flame oxidation system.


Supplemental Fuel
Dilute Fume if Required
Odorous Fumes
Fuel Combustion ~1370°C
Slipstream
Mixing of Fume and Hot Combustion Gases ~650-800°C
Bypass Stream
Retention of Fumes at High Temperature for Sufficient Time
Clean Air

An example of the use of combustion systems for odor control is at the 760,000 m^3/d (200 mgd) Metro Plant in Minneapolis, Minnesota. The sludge handling scheme consists of gravity and dissolved air flotation thickening, sludge storage, low pressure oxidation (LPO), plate and frame filter presses, and multiple hearth incineration. There are four decant tanks for the LPO system and odors from the tanks are contained for destruction or treatment. The Metro Plant has several options for treatment of the odors from the decant tanks.

The first option is to use the odorous air as secondary combustion air for the incineration process. Exhaust air from the decant tanks is combined and diluted with the air from the solids handling building and fed to the incinerator. Experience has shown that the higher the temperature in the hearths, the better the removal of odors. The Minnesota Pollution Control Agency restricts the plant to an odor concentration of less than 150 odor units at the plant boundaries. If the incinerator is down, the plant can use the originally designed wet scrubber followed by activated carbon contactors. Problems have been experienced with the carbon regeneration which did not remove all the organic ketones contained in the decant tank vapors. As a result, the ketones remaining on the carbon caused spontaneous combustion in the carbon bed during backwash with steam at 93° to 121°C (200° to 250°F).

The city tried using a spray chamber absorption system but was not satisfied with the results. An effluent gas with a sweet odor was produced, apparently the result of the organics not being completely removed.

c. Costs

Typical costs for direct flame incinerators and catalytic incinerator systems for odor control are shown in Tables 4-12 and 4-13, respectively (36). The cost of thermal and catalytic incinerators is a function of the volumetric air flow rate and heat content of the gas, and various design factors, including materials of construction, type of refractory, level of control and required air residence times.

Budget level cost estimates (+30 percent, −15 percent) have been developed to show typical costs of combustion systems for odor control. In practice, detailed cost estimates must be developed which take into account the specific site conditions, actual concentrations and

Table 4-12. **Typical Costs for Thermal Incinerators for Odor Control[a] (1984 $) (37)**

Air Residence Time	Capital Cost[b], $ Air Flow m³/min	
	28	280
sec		
0.2	19,800	31,500
0.5	24,800	41,300
1.0	31,400	51,200
2.0	41,300	72, 600

[a] Assumed H_2S inlet concentration = 20 ppm; outlet concentration <1 ppm.

[b] Includes cost of incinerator, incinerator base, fan, motor, starter, integral ductwork, controls, instrumentation, and refractory linings. Does not include heat exchanger.

Table 4-13. **Typical Costs for Catalytic Incinerators[a] (1984 $) (37)**

Type of Incinerators	Capital Cost[b], $ Air Flow m³/min	
	28	280
Package units	26,100	42,500
Custom units without heat exchange	35,300	70,300
Custom units with heat exchange	60,900	97,700

[a] Assumed H_2S inlet concentration ppm; outlet concentration <1 ppm.

[b] Includes catalyst bed, refractory lining, preheat burners, ductwork, fan, controls, installation.

variability of odorous contaminants, necessary ventilation rates, and objectives in odor control.

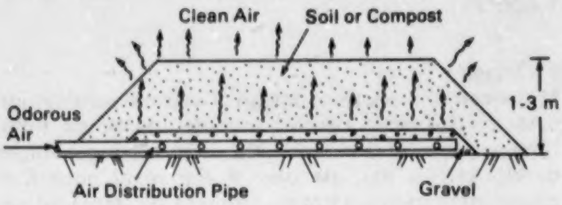
4.3.2.6 Soil/Compost Filters

Soil and compost filter beds have been successfully used to remove odors from relatively small volumes of air containing odorous compounds (37-40).

They can be used as a primary odor control system or as a polishing step for exhausts from other odor control systems such as wet scrubbers. Contaminated gas is introduced into a bed of soil or compost through perforated pipe located near the bottom of the bed. As the gas passes upward through the bed, the soil substrate sorbs and oxidizes odorous gases such as H_2S, SO_2, and NH_3. The oxidation products are CO_2, H_2O, sulfate, and nitrate.

The soil provides sorptive surface area, structural support, and nutrients and water to support bacteria which can biologically oxidize certain odorous compounds. Several different configurations of soil or compost filters have been employed. A schematic diagram of a soil/compost filter system is shown in Figure 4-17.

Figure 4-17. **Soil/compost filter for odor control.**



Soil filters require moisture for the sorption and reactions of both organic and inorganic gases. To maintain an optimum moisture content, watering of the bed may be required. Too little water may cause reduced microbial activity, lower sorption capacity, and cracking of the soil which may lead to short-circuiting and release of odors. Too much water may cause development of anaerobic conditions and loss of filter function. In very moist climates, it may be necessary to provide an underdrain for removal of excess water, or to cover the filter bed.

Temperature of the bed is important to filter performance. Temperature of the bed is affected by the temperature of the inlet gas and degree of microbial activity. While an active microbial population is desirable, very high soil temperatures decrease the capacity for physical adsorption of the odorous gas.

Soil filters typically use sandy loam, compost, or mixtures of soil and peat moss, with depths of 1 to 3 m (3 to 10 ft). Performance is dependent on numerous factors, including the type and concentration of the odorous compounds to be removed, characteristics of the filter media (organic content, bulk density, porosity, etc.), moisture content of the bed, temperature, bed depth, and time in service.

A full-scale compost filter system was constructed in 1978 at Moerewa, New Zealand to treat odors from a treatment facility handling wastes from a rendering plant (40). Approximately 1 m (3 ft) of sludge-derived compost was used as the filter media in a 7 m x 6 m (23 ft x 20 ft) bed. Perforated plastic pipe was used to distribute the odorous air into the filter bed. The bottom of the bed was sloped slightly to allow collection and removal of excess water. The system was designed to treat 15 m³/min (530 cfm) of air containing H_2S concentrations of up to 1,000 ppm by volume. Results from 4 months of testing of the full scale odor removal system are shown in Table 4-14. As can be seen from the data, the filter bed was successful in removing the high H_2S concentrations in the influent air.

Table 4-14. Performance of Compost Filter for H_2S Odor Control at Moerewa, New Zealand (40)

Ambient Air Temperature	Internal Temp. of Filter Bed	H_2S Concentration						Average H_2S Removal (infl.-effl.)
		Influent		Effluent[a]		Atmosphere[b]		
		Max.	Min.	Max.	Min.	Max.	Min.	
°C	°C	ppm						%
7-13	29-57	660	37	0.08	0.00	0.32	0.00	99.97
12-28	33-58	600	9	0.11	0.01	0.18	0.00	99.79
16-29	32-55	820	49	0.19	0.00	0.24	0.00	99.93
10-27	21-46	703	155	0.29	0.00	0.08	0.00	99.96
10-28	24-51	487	112	0.10	0.00	0.10	0.00	99.98
10-29	24-50	406	219	0.11	0.00	0.08	0.00	99.98
16-25	34-46	669	139	0.10	0.01	0.10	0.00	99.98
19-28	20-47	320	75	0.12	0.00	0.04	0.00	99.98
8-25	21-52	400	84	0.19	–	0.04	0.00	99.91
7-23	11-32	195	69	0.00	0.00	0.04	0.00	100.00
9-23	17-40	535	70	0.00	0.00	0.00	0.00	100.00
5-21	15-40	99	23	0.02	0.00	0.00	0.00	99.99

[a] 40 mm above surface.

[b] 2 m above surface.

Pilot studies on soil beds for H_2S control were conducted at the University of Washington (38). Major conclusions from the study were:

1. Moist loam soils were found to have excellent possibilities in the efficient and inexpensive removal of undesirable odorous gases from air streams. Sand and clay were less satisfactory soils.
2. The effects of ion exchange, chemical composition, or oxidation and soil or water adsorption were negligible, the primary odor reduction mechanism being through the action of microorganisms.
3. Conditions should be maintained in an optimal range for bacterial growth. The soil should be kept moist and warm.
4. Over a 3-month test period, H_2S gas concentrations of 15 ppm at a loading rate of 0.11 $m^3/min/m^2$ (0.35 cfm/ft^2) of soil surface were reduced to imperceptible levels in 81 cm (32 in) of soil.
5. Effectiveness of the soil beds for removal of H_2S did not diminish during the 3-month test period.

a. Equipment

Equipment requirements for a typical soil/compost odor filter are minimal. A simple system consists of a perforated pipe network surrounded by gravel to prevent plugging of the perforations, and overlaid with a suitable media such as sandy loam or compost. Air piping should be constructed of acid-resistant materials such as PVC. Valves may be employed to direct odorous air to certain portions of the filter bed, or to bypass parts of the beds to allow maintenance. For moist climates where soil saturation may be a problem, an underdrain system may be used to remove excess water, or the beds may be covered to prevent direct infiltration from rainfall. In dry climates, periodic watering may be required to maintain a suitable moisture content in the bed. This may be accomplished manually or through the use of an automated watering system.

b. Design

Design of soil or compost filters depends on the nature and concentration of odorous compounds, air flow rates, target concentrations in exhaust gas, and characteristics of the soil or compost media. Designs vary from simple filter beds with networks of perforated PVC pipe to more elaborate systems with concrete berms, underdrain systems to collect excess water, and automated watering hardware. Most designs employ a gravel pack surrounding the gas conduit. Gravel suitable for this application might have diameters ranging from 2 to 4 cm (0.8 to 1.6 in). Depth of gravel depends on the size of the gas conduit, but is typically 0.2 to 1 m (0.8 to 3 ft). A permeable barrier such as marsh hay, screens, or synthetic fabric is placed above the gravel to prevent soil fines from infiltrating into the gravel and air distribution system. The gravel is then covered with 1 to 3 m (3 to 10 ft) of loosely packed sandy loam or compost.

The air loading rate (volumetric rate of air flow per unit of horizontal filter area) to the filter bed should be determined empirically. Loading rates for experimental and full-scale facilities have ranged from 0.11 to over 1.0 $m^3/min/m^2$ (0.35 to 3.3 cfm/ft^2) (37-40). For high concentrations of odorous compounds or compounds which oxidize slowly, loading rates on the low end of the above range are employed. Higher loading rates would be appropriate for air contaminated with only 10 to 15 ppm of H_2S.

Because of the many variables affecting design and performance, it is recommended that pilot studies be conducted before designing full scale facilities. Such studies can be conducted at little cost in order to collect sufficient data to establish design criteria.

c. Example—Columbus Southwesterly Compost Facility, Columbus, Ohio
This compost facility, which handles about 182 metric tons (200 tons)/d of raw and waste-activated sludge, had odor complaints from the community and a court order to cease the odor nuisance. Sludge is hauled as a 16-percent TS centrifuge cake to the facility from the Southerly Wastewater Treatment Plant (WWTP) in Columbus, Ohio.

Most of the odors emanated from the mixing operation, where sludge and wood chips were combined. Therefore, a 30 m x 60 m x 9 m high (100 ft x 200 ft x 30 ft) pre-engineered metal building, with an air handling system, was erected to contain this operation. The air handling system provides five to six air changes/hour. The air is exhausted through a network of plastic pipes into a "bio-filter" which contains about 0.8 m (2.5 ft) of cured, composted material.

This project has been successful, but the facility generated odors from other sources, particularly the composting and curing piles. It was identified that the odors only occurred when insufficient air (oxygen) was supplied to these operations. In fact, plans have been made to add air to the curing stages (30-days storage). By keeping the piles aerobic, objectionable odors will be reduced.

d. Costs
Table 4-15 presents estimated costs of compost filters for odor removal at air flow rates of 28 m^3/min (1,000 cfm) and 280 m^3/min (10,000 cfm).

The costs presented are budget level estimates (+30 percent, -15 percent). Actual costs would depend on site conditions, availability and costs of compost, and the degree of sophistication of the design. Provisions for concrete berms, underdrains, covers, or sprinkler systems would increase the cos

Table 4-15. Estimated Costs of Compost Filters for Odor Control[a] (1984 $)

Design Air Flow	Capital Costs[b]	Annual Cost of Compost Replacement
m^3/min	$	$/yr
28	9,700	300
280	89,000	3,000

[a] Assumed conditions:
1. Continuous operation
2. Inlet H_2S = 20 ppm
3. Outlet H_2S = <1 ppm
4. Loading rate of 0.2 m^3/min/m^2 (0.66 cfm/ft^2)
5. Compost replaced after 5 years

[b] Including excavation, piping and installation, fan, compost, backfill and grading.

4.3.2.7 Existing Biological Stabilization Processes
In several cases, odorous air has been collected and piped into the bottom sections of trickling filters and activated sludge basins for effective odor control.

Introducing air into the bottom of a trickling filter results in countercurrent flow similar to that in a wet scrubber system, and can be a cost-effective technique for odor control. This method has been successfully used at Beaumont, California; Avila Beach, California; Stockton, California; Sydney, Australia, and elsewhere (39).

In some cases where the influent to the trickling filter contains dissolved sulfides, introduction of odorous air into the bottom of the filter may strip out H_2S from the liquid, and discharge malodorous gases at the top. At Palm Springs, California, two perforated pipes discharged waste digester gas and sewer air to a point mid-depth in the 2.9-m (9.5-ft) deep rock filter (39). Air was drawn down through the filter in sufficient volumes to maintain a constant downward air velocity that prevented escape of odors at the top.

Activated sludge basins have been used for odor control by introducing malodorous air into the inlet side of the blowers. Biological deodorization occurs in the overlying aerated liquid in the aeration basin. This method was tested at pilot-scale and subsequently adopted for use at the Los Angeles Hyperion plant. This method is most applicable for controlling H_2S odors, because sulfide is rapidly oxidized in the aerobic mixed liquor. Other odor compounds may not react this way, and although some dissolution may occur, odors may not be removed completely (39).

a. Equipment
Equipment requirements are minimal, and consist mostly of piping to convey the odorous gases to the biological process. Where trickling filters are used for biological odor control, an auxiliary blower may be required to force the gas through the media. For an activated sludge system, existing blowers can be employed to inject the odorous gas into the basin.

Piping should be corrosion-resistant to prevent attack by H_2S gas and sulfuric acid which may form from the oxidation of sulfides. H_2S in the input air generally does not harm blowers, since the high temperature prevents condensation. Downstream piping may be affected, however, due to cooling, condensation of moisture, and production of sulfuric acid. Underdrains in trickling filters may also be affected, and should be constructed of corrosion-resistant materials.

b. Design

Effectiveness of the technique depends on the nature and concentration of the odor, wastewater characteristics in the stabilization process, volume of air to be treated, and the area and depth of the stabilization units. For trickling filters, contact time of the gas with the wastewater and media is important. Minimum contact time of 8 to 10 seconds has been found effective (39).

Piping of odorous air to activated sludge basins should consider dispersal of the air over the entire basin to achieve initial dilution and allow rapid oxidation of sulfides in the mixed liquor.

Because of the lack of design criteria and the wide variability in gas characteristics and process designs, pilot studies are recommended prior to implementation of this odor control alternative. Such studies can be conducted relatively inexpensively to determine the effectiveness at various air flow rates.

c. Example—Kalamazoo WWTP, Kalamazoo, Michigan

At the 204,000 m^3/d (54 mgd) Kalamazoo WWTP, odors from all of the covered plant processes are collected in fiberglass piping and directed into the activated sludge air blowers and diffused into the powdered activated carbon secondary aeration basins. The processes from which air is collected include grit chambers, primary clarifiers, thickeners, screen room, low pressure oxidation decant tanks, vacuum filter building and vacuum pumps. The air is directed to a block-constructed plenum and serves as make-up air to the centrifugal air blowers.

Air from the decant tanks is first run through a wet scrubber before being combined with the other odorous gases. In addition, odorous air from the decant tank will be used as combustion air for the incineration process.

4.3.3 Odor Modification, Counteraction, and Masking

There has been a great deal of confusion as to the purpose and effectiveness of using various chemical agents to disguise odors, reduce their intensity, or render them less offensive. The following definitions are therefore provided (41)(42).

Odor modification is the name given to a process whereby two substances of given concentration are mixed, with the resultant odor of the mixture being less intense than that of the separate components, and in some cases imperceptible.

Odor counteraction is the name given to the phenomenon whereby odor intensity is reduced by adding non-chemically reactive odor agents to a malodor. Examples of pairs of odiferous gases that, in certain proportions, result in the mixture being odorless or nearly so are: ethyl mercaptan and eucalyptol; skatole and coumarin; and butyric acid and juniper oil.

Odor masking is the name given to a method by which the quality of a malodor is overcome by mixing it with a substance having a strong, pleasant odor. Thus, the pleasant odor effectively masks the malodor, resulting in a less objectionable odor.

Counteraction agents are available to counteract the following odor types (3):

1. Phenolic
2. Aldehyde
3. Amine
4. Solvent
5. Mercaptan
6. Aromatic
7. Organic, fatty acid

Typical masking agents include heliotropin, vanillin, eugenols, benzyl acetate, and phenylethyl alcohol. These are organic aromatic compounds that can be effective as short-term or emergency solutions to severe odor problems.

Odor modification, counteraction, and masking have met with variable success at wastewater treatment plants. They have often been applied as interim solutions, or for short duration during periods of odor emissions. In some cases where odor masking agents have been used, the resulting odors have been considered equally unpleasant. The problem is often further complicated by the subjective human response to odors. In general, prevention of odor emissions and positive control and removal of the odor-causing substance(s) are preferred alternatives for effective odor control.

A more detailed discussion of the theory of odor modification is contained in Reference 43.

4.3.3.1 Equipment

Equipment requirements for application of odor modification, counteraction, and masking agents are generally minimal and depend on the method of addition. Application techniques include the following (41):

1. Direct addition to the odorous material
2. Spraying
    a. hand spraying
    b. automated spraying
    c. stack spraying
    d. aerial spraying

3. Metered feeding to the liquid stream
4. Dispenser feeding to an air stream

One of the most common application techniques used at wastewater treatment plants is automated atomized spray systems located around the periphery of the plant for dispersion of masking agents.

4.3.3.2 Design

Designing for use of an odor modification agent requires characterization of the odor and screening of various chemical agents to determine their effectiveness. This requires subjective testing, as with an odor panel.

Important considerations when evaluating use of odor modification, counteraction, and masking agents are (3)(41):

1. ANY GAS OR VAPOR THAT MAY BE TOXIC (SUCH AS H_2S) SHOULD NOT BE DISGUISED.
2. Odor modification should not be used as a substitute for good housekeeping.
3. If possible, odors should be controlled at the source.
4. Odor modification, counteraction, and masking agents must conform to local, state, and federal specifications and regulations.
5. Odor modifying chemicals should always result in a lowering of the olfactory intensity.

4.4 Corrosion in Wastewater Treatment Plants

The physical and chemical mechanisms of corrosion are fully discussed in Section 2.4. The following discussion will focus on the conditions in wastewater treatment plants that lead to corrosion of materials.

4.4.1 Continuous or Intermittent Submerged Exposure

These conditions include those in which the component is subjected to direct contact with wastewater, wastewater sludge, or various process sidestreams on either a continuous or intermittent basis.

Plant components or structures at the waterline are subject to the most severe exposure in terms of potential corrosion, since the structure is alternately subjected to wet and dry conditions as well as fluctuations in ambient temperatures. Water itself may be destructive because, in the presence of salts, it acts as an electrolyte. Water may hydrolyze certain paint components, which decreases both the strength and adhesion of the coating. In addition, it may decrease the resistance of the coating to passage of oxygen and other gases such as H_2S, which may be corrosive to the underlying material. Oils, greases, and soaps which typically accumulate at the waterline of wet wells, holding tanks, and aeration basins may also present a problem because these materials contain solvents which may soften the paint or coating and make it more susceptible to abrasion or rupture.

Paints and protective coatings at the waterline are also subjected to physical forces, such as those stresses formed by alternately wetting and drying, heating and cooling, and freezing and thawing. These actions may be highly destructive. In addition, sunlight may affect certain organic coatings and reduce their effective life.

Iron and steel do not readily deteriorate when completely submerged in wastewater due to the formation of a protective glassy iron oxide film on the surface. However, when exposed to the atmosphere, the iron oxide coating quickly comes loose to expose fresh metal, and the corrosion process is continued.

Metallic zinc coatings on steel aeration basins or on metal components in aeration basins are subject to deterioration, apparently due to the presence of carbon dioxide resulting from biological oxidation, and from the high DO content (44).

The presence of high DO concentration is also cited as the reason for accelerated deterioration of materials in pure oxygen activated sludge plants. This was the subject of an EPA-sponsored research project in which the resistance of various materials was evaluated under the conditions found in such plants (45).

4.4.2 Moist Atmosphere Exposure

Enclosed wet wells, grit and screen chambers, holding tanks, and buildings housing wastewater or sludge processing equipment are often potential sites for corrosion due to the presence of moisture as well as corrosion-inducing gases such as H_2S. In such enclosures, moisture tends to condense in a film on cold surfaces such as walls, handrails, pumps, conduits, and windows. The moisture absorbs oxygen and other gases which may result in production of very corrosive condensates.

It has been found that H_2S gas can penetrate many paints and coatings. Upon reaching steel, H_2S reacts directly to form iron sulfide. Buildup of corrosion products and gas may blister and rupture the coating, allowing more rapid corrosion of the freshly exposed material. H_2S may also be oxidized to sulfuric acid in the presence of oxygen and moisture, allowing the acid to attack the coating or penetrate the surface through pinholes or defects. The acid will then vigorously attack the underlying material, allowing corrosion to spread laterally beneath the coating.

Another problem with exposure to materials in moist atmospheres is the physical effects of stresses in the paint or coating due to fluctuations in moisture and temperature. Such variations result in expansion and contraction of the coatings which may result in rupture of the protective film.

Moist atmosphere exposure is perhaps the single most destructive condition for protective coatings and other materials encountered in a wastewater treatment plant (45).

4.4.3 Dry Atmosphere Exposure

Relatively dry atmospheres inside buildings are generally not conducive to corrosion unless H_2S gas is present. H_2S may attack metal directly, and this can present a problem with oxidation of electrical contacts and other metal surfaces. In addition, exposure of certain paint pigments to H_2S may result in discoloration.

4.4.4 Outside Weather Exposure

Normal exposure to ambient weather conditions can result in corrosion and deterioration of paints and protective coatings due to the effects of sunlight, humidity, temperature fluctuations, salt, and abrasion. Although such problems may be found at any building or structure, the effects may be accelerated by the presence of H_2S or other corrosive chemicals associated with sewage treatment works.

Effects of exposure to ambient conditions include aging, cracking, and discoloration of paints due to sunlight; deterioration of metal surfaces by salt air; wear of paints and coatings by abrasion from blowing dust and sand; deterioration of concrete by freeze-thaw cycles; and discoloration of paints and corrosion of metal surfaces by H_2S.

4.4.5 Miscellaneous Exposure

Materials used in certain unit processes or process components may be subjected to special conditions which foster corrosion. Examples include physical-chemical treatment processes, anaerobic digester heating systems, plant boilers and steam piping, and thermal sludge conditioning systems.

Physical-chemical treatment system components are often subject to corrosion and scaling from the use of chemicals such as lime and ferric chloride. Scaling may be a serious problem with the use of strong alkalies due to precipitation of calcium carbonate scale on metal or concrete surfaces at elevated pH. Ferric chloride is a very corrosive material which will depress the pH of wastewater, resulting in a more aggressive water which may attack both concrete and metallic components.

Boilers and heat exchangers used in digester heating may also be subject to corrosion and scaling due to calcium precipitation and galvanic currents established from differences in temperature.

Stress corrosion and surface pitting of stainless steel heat exchangers in thermal sludge conditioning units have been reported at chloride concentrations exceeding 500 mg/l. Use of caustic soda was adopted at Green Bay, Wisconsin to increase the pH of the sludge from 4.2 to 5.5 in order to control the rate of corrosion in the thermal sludge conditioning system. Formation of calcium oxalate scale required flushing of the system with a 5-percent nitric acid solution every 500 operating hours (46).

4.5 Corrosion Control Techniques at Existing Wastewater Treatment Plants

A variety of methods is available for control of corrosion in wastewater treatment plants. The following discussion presents the most common alternatives employed for corrosion control.

4.5.1 Chemical Addition

Chemical addition has been employed to control sulfide-induced corrosion in wastewater treatment plants. The action of chemicals on H_2S results in oxidation, precipitation, or inhibition of sulfide formation. Chemicals previously discussed in this chapter include hydrogen peroxide, metal salts, nitrates, and chlorine. These chemicals have been found effective for sulfide control; however, not all corrosion problems are sulfide-induced, and use of such chemicals may not be appropriate for control of corrosion due to other conditions or agents. For example, for low pH waters, addition of alkalies such as caustic soda can elevate the pH to a neutral range, greatly reducing the rate of acid-induced corrosion of concrete and metal surfaces.

4.5.2 Cathodic Protection

Cathodic protection has been successfully used for corrosion control of iron and steel components, such as clarifiers, aeration tanks and sludge digesters, in both water and wastewater treatment systems. Cathodic protection is defined as "the reduction or prevention of corrosion of a metal surface by making it cathodic, for example, by the use of sacrificial anodes or impressed currents" (44). If an electrical current from any source is impressed on a corroding metal in sufficient amounts to neutralize the corrosion currents, corrosion will cease in that particular area. In normal applications of cathodic protection, the metal to be protected is electrically connected to the negative terminal of a current source, and the positive terminal is connected to an anode in the corrosive electrolyte. Current from the anode passes through the electrolyte to the metal, making it cathodic and reversing the current at the anodes of localized galvanic cells which have been established in the metal. The source of current is typically a rectifier which supplies low voltage direct current of several amperes.

Cathodic protection systems using sacrificial anodes must use anodes that are more active in the galvanic series than the metal to be protected. For iron or steel protection, aluminum, zinc, and magnesium could theoretically be used; however, magnesium has the highest electrical potential and is most often used (44). Other anode materials include niobium, titanium, and high-silicon cast iron.

The applied voltage required to give adequate corrosion protection is determined by measuring the electrical potential of the structure to be protected. This is the key criterion in assessing the effectiveness of cathodic protection and in designing an appropriate system.

Because success of cathodic protection systems has been variable, a competent corrosion control engineer should be consulted prior to design and installation of such a system.

4.5.3 Ventilation and Heating

Ventilation and heating of enclosed spaces for humidity control have been successfully used to prevent condensation of corrosive vapors on components and structures in wastewater treatment facilities. In most cases, moisture must be present for corrosion to occur. Adequate provision of heating and ventilation systems in enclosed spaces such as screen and grit buildings, pumping rooms, and pipe tunnels can reduce or eliminate condensation of moisture, and thus greatly reduce the rate of corrosion and increase the life of protective paints and coatings. Recommendations for design of ventilation and heating systems for enclosed spaces are provided in Chapter 6.

4.5.4 Materials Selection

When deteriorated components require replacement, consideration should be given to the conditions which resulted in the deterioration and the use of materials which will be more resistant to those conditions. Corroded appurtenances such as submerged sludge collector chains in rectangular clarifiers, general facility decking and walkways, handrails, ventilation ducts and other components can generally be replaced with materials such as plastic, fiberglass or stainless steel which are more resistant to the corrosive exposures experienced in those applications. A detailed discussion of materials selection in the design of facilities for corrosive environments is presented in Chapter 6.

4.6 Case Histories

4.6.1 Odor Control Using Wet Scrubbers

The Aptos wastewater transmission system, operated by the Santa Cruz County (California) Sanitation District, is 13.7-km (8.5-mi) long, employing four regional pump stations ranging in capacity from 11,000 m^3/d (2.9 mgd) to 21,000 m^3/d (5.5 mgd). Transmission lines are constructed of polypropylene, with diameters ranging from 38 cm (15 in) to 122 cm (48 in). Detention times in the system can be as long as 24 hr. Two of the four pump stations are located adjacent to beaches, one at the edge of a shopping center, and one directly behind an exclusive restaurant. The Aptos wastewater transmission system was completed in 1979 at a cost of $8,500,000, and included odor control equipment at the lift stations and in the force mains.

The odor control equipment at the lift stations consisted of wet scrubber reactors utilizing sodium hypochlorite (NaOCl) as the scrubbing solution. Each 0.6-m^3 (20-cu ft) reactor contained a packed bed of 2.5-cm (1-in) diameter random-dumped plastic rings. The NaOCl solution was continually recirculated at a preset, manual rate. Make-up NaOCl was required to maintain the concentration at the desired level. Design requirements called for 95 percent removal of an anticipated maximum H_2S concentration of 10 ppm in the wet well ventilation exhaust.

In addition to wet scrubbers, air injection equipment was installed at various locations along the force main to prevent anaerobic conditions and to minimize sulfide buildup and odor release.

Within 1 year after start up of the transmission system, both the air injection and wet scrubber system experienced operating and performance limitations, resulting in frequent odor complaints. Excessive pressures developed in the force main resulting from air injection, causing air relief manholes to blow off. In the wet scrubbers, higher than anticipated H_2S levels and insufficient contact of the H_2S with the NaOCl solution resulted in incomplete reaction of the odorous gases, even at maximum NaOCl concentrations. Clogging of nozzles and pump impellers occurred, apparently due to formation of salts at NaOCl concentrations greater than 5 percent by weight. Daily monitoring of the chemical concentrations was required to maintain the desired NaOCl levels.

The district chose to install a proprietary odor control system as a demonstration unit at the New Brighton Beach pump station. The system was wet scrubber type of unit which eliminated the need for packing material by generation of a fog of micron-sized droplets in the contact chamber that provided much greater surface area (6,000 m^2/m^3) than conventional packing materials. This was accomplished by using compressed air in conjunction with specially designed nozzles. Water softeners were required to prevent scale formation and nozzle clogging. The demonstration unit employed a 9.6-m^3 (340-cu ft) stainless steel contact chamber, allowing contact times of up to 30 seconds. The scrubber liquid employed was sodium hypochlorite (for H_2S oxidation) and sodium hydroxide (for pH control). Sodium hydroxide feed rates were automatically adjusted based on the pH of the spent scrubber liquid. As opposed to conventional wet scrubbers which continually recirculate the scrubber liquid, this system used a once-through process with no recirculation of scrubber liquid.

The objective of the new odor control system called for a removal efficiency of 90 percent of an anticipated maximum influent H_2S level of 30 ppm. The following design criteria were developed for the demonstration unit:

Removal efficiency = 90 percent
Air flow (min) = 28 m^3/min (1,000 cfm)
Chemical flow = 7.6 l/min (2 gpm)
Compressed air = 2.2 m^3/min @ 550 kPa (80 cfm @ 80 psi)
Soft water flow (max) = 7.6 l/min (2 gpm)
Reaction time (min) = 10 sec

During the initial performance tests, inlet H_2S concentrations up to 57 ppm were reported, which were beyond the design capacity of the system. However, relatively simple modifications to the system allowed acceptable performance at the higher loadings. These modifications consisted of reducing the exhaust fan speed (increasing contact time), and increasing the nozzle size.

Final performance tests were conducted at air flows of 28 m^3/min (1,000 cfm), liquid (chemical) flows of 2.6 l/min (0.7 gpm), and inlet H_2S levels of 30 to 100 ppm. The results of these tests are summarized in Table 4-16. As can be seen from the data, performance was excellent, with removal efficiency averaging approximately 95 percent.

Table 4-16. Results of Wet Scrubber Performance Tests for H_2S Control, Santa Cruz County, CA

Test No.	Average Inlet H_2S	Outlet H_2S	Average Removal Efficiency
	ppm	ppm	%
1A	31	1.0	97
1B	42	1.8	96
2A	40	1.6	96
2B	44	2.0	95
3A	56	2.8	95
3B	32	0.8	98
4	38	1.0	97
5	43	0.4	99
6	92	5.4	94
7	54	4.0	93
8	57	3.8	93
9	45	1.6	96
10	55	4.4	92
11	43	2.3	95
12	166	24	86
–	100	10	90
–	58	1.6	97

The outlet gas from the new unit is now introduced into the older, packed scrubber (using clear water as the scrubbing solution) for removal of any chlorine odors. The system has operated satisfactorily for over 1 year. NaOCl consumption is approximately 76 l (20 gal)/day. Based on successful performance at the New Brighton Beach pump station, additional units have been planned for two other lift stations experiencing odor problems[1]

4.6.2 Odor Control With Wet Scrubbers and Activated Carbon

Severe odor problems existed at several pumping stations in one of two major sewage collection systems serving the city of Tampa, Florida. Wastewater flows in the system averaged only 10 to 15 percent of design flows, resulting in detention times in the force mains of 5 to 6 hours. Gentle slopes and warm temperature combined to make conditions conducive to sulfide generation. H_2S released from pumping stations created a severe odor nuisance in adjacent residential neighborhoods. H_2S concentrations in one pumping station were as high as 200 ppm (47).

Chemical addition to the wastewater to oxidize or inhibit sulfides was rejected as being too costly or inappropriate for the specific conditions. Due to the irregular profile of the sewer, air and oxygen addition was ruled out because of the potential for gas locking. It was decided, therefore, to investigate treatment of the H_2S-laden air. Design criteria for the treatment alternatives were based on the following:

1. Continuous ventilation of wet wells at the rate of 12 air changes/hour.
1. Intermittent ventilation of wet wells at the rate of 24 air changes/hour.
3. H_2S concentrations to be treated:
   a. 10 ppm H_2S—low average concentration
   b. 25 ppm H_2S—moderate average concentration
   c. 50 ppm H_2S—observed average concentration
4. Design outlet H_2S concentration of 1 ppm.

A wide range of alternatives was reviewed. Based on a cost-effectiveness analysis of these alternatives, a system consisting of wet scrubbers using sodium hypochlorite followed by NaOH-impregnated activated carbon adsorbers was selected. Four of these systems were installed in 1981 and 1982.

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[1] Personal Communication: John V. Nutt and Martha Shedden, Santa Cruz County Sanitation District, California.

Design criteria and operating data from one of these installations are presented in Table 4-17. Due to equipment malfunctions, including rectifier overloads, and recirculation pump and cooling fan failures, the wet scrubber did not perform as designed, removing an average of about 70 percent of the incoming H_2S. As a result, mass loadings of H_2S to the activated carbon units exceeded design values, requiring more frequent regeneration and replacement of the carbon than originally anticipated. It should be noted that when all equipment was operating satisfactorily, the wet scrubber system was effective, removing 85 to 95 percent of the incoming H_2S.

Table 4-17. Design and Performance of Wet Scrubber/ Activated Carbon Odor Control System at Tampa, FL (47)

Operating Parameter	Design Value	Initial Operating* Data Value
NaOCl Wet Scrubber Inlet Annual Average H_2S Concentration - ppm	50	49.1
NaOCl Wet Scrubber Outlet Annual Average H_2S Concentration - ppm	5	14.5
NaOCl Wet Scrubber Average H_2S Removal - Percent	90	70
NaOH Impregnated Activated Carbon Annual Average Removal - Percent	100	100
NaOH Impregnated Activated Carbon Virgin Run - Days	730	180

*Includes abnormal conditions due to equipment startup variations and malfunctions.

Although the combined wet scrubber-activated carbon system was effective in removing H_2S from the contaminated air when all equipment was functioning, additional problems developed. A sweet, ripe-olive odor was detected at the outlet of all four odor control systems. Although H_2S was still being removed, it was postulated that another organic odor was breaking through the activated carbon adsorbers. A program was initiated to determine the source of this odor and to analyze alternatives for its removal.

Another problem which occurred was the presence of hydrochloric acid vapors in the discharge ductwork of the hypochlorite scrubbers. These vapors condensed in the ductwork and the activated carbon columns, resulting in pitting of the stainless steel dampers and corrosion of the duct silencers. In addition, this appeared to adversely affect the adsorption capacity and expected life of the caustic-impregnated carbon.

The problem was analyzed and found to result from excess NaOCl (fed at a constant rate) during periods of low H_2S concentrations, and from variation in pH due to feeding NaOH at a constant rate. Automatic controllers and pH monitors were proposed to control NaOCl and NaOH feed rates (47).

4.6.3 Odor Control Using Activated Carbon

In Ocean County, New Jersey, odors were being released from pumping stations in large collection systems serving three regional treatment plants (48). Some of the pumping stations were located as close as 15 m (50 ft) from residences, and were the source of numerous odor complaints.

To solve the odor problem, 14 activated carbon adsorption systems were installed at various pump stations where odor problems existed. Ozone systems had been installed, but it was found that operation and maintenance requirements of these highly instrumented systems were too complex for the operating staff. Wet scrubbers were used for the less sensitive pump stations, but it was felt that activated carbon would be more effective, particularly for those pump stations handling strong industrial wastes.

The 14 adsorbers were all of reinforced fiberglass construction, with diameters ranging from 1.2 to 3.7 m (4 to 12 ft). Air handling capacities of these units varied from 28 to 250 m^3/min (1,000 to 9,000 cfm). The smallest adsorber contained 570 kg (1,250 lb) of granular activated carbon, while the largest contained 4,770 kg (10,500 lb). Total carbon content of the 14 adsorbers was approximately 36,400 kg (80,000 lb).

Thirteen of the 14 adsorbers used a caustic-($NaOH^-$) impregnated carbon specifically designed for H_2S removal. The other used a high surface area carbon media that has a strong affinity for alcohols, chlorinated hydrocarbons, esters, ketones, hydrocarbons, and aromatics. The activated carbon systems have been found effective in controlling odors from the pump stations.

At the Loxahatchee River Wastewater Treatment Plant in Jupiter, Florida, an activated carbon system is used to control odors from the master lift station. Periodic checks of the influent H_2S levels to the system indicate influent H_2S concentrations of 0 to 60 ppm.

A 1.22-m (4-ft) diameter, single bed adsorber is employed using NaOH-impregnated carbon. Air flows through the unit are approximately 0.33 m^3/s (700 cfm). The unit began operating in December 1981.

The installed capital cost of the system was approximately $25,500 in 1981. Operation and maintenance costs are shown below.

Virgin carbon lasts approximately 10 months before breakthrough occurs. The carbon is regenerated once using 1,130 (300 gal) of 50 percent NaOH solution. The unit contains 613 kg (1,350 lb) of activated carbon. After regeneration, the carbon lasts approximately 6 months, at which time the spent carbon is replaced with virgin carbon. Thus, the system operates on a 16-month cycle.

On a 16-month basis, the materials cost for carbon regeneration and replacement is as follows:

Virgin carbon:	$7.33/kg x 613 kg	= $4,495
NaOH regeneration:	$0.17/l x 1,130 l	195
Total for 16 months		= $4,690

The city estimates that operation and maintenance of the system requires 8 man-days/yr of labor. The total annual operating and maintenance (O&M) costs are estimated to be:

Carbon and NaOH (12 mo)	$3,520/yr
O&M labor—8 days at $160/day	1,280/yr
Power costs @ $.07/kWh	600/yr
Maintenance mat'ls and supplies	180/yr
Total Annual O&M Costs	$5,580/yr

The system has been successful in controlling odors from the master lift station. Odors are not detectable until breakthrough of the carbon occurs. The need for regeneration of the carbon is determined based on sensory perceptions from city personnel or frequency of odor complaints from neighbors. Monitoring of inlet and outlet H_2S concentrations is not routinely conducted.

4.6.4 Odor Control Using Direct Combustion

The city of North Olmstead, Ohio operated a Zimpro low oxidation process for conditioning gravity-thickened primary and waste-activated sludge from its 34,000-m^3/d (9-mgd) activated sludge plant. All sludge from the plant was gravity thickened, heat treated at 132°C (270°F) and vacuum filtered prior to disposal. Odors from the heat treatment processes were a severe problem and created nuisance complaints because of the proximity of the plant to surrounding homes (50). The original Zimpro installation included a catalytic burner for treatment of odorous gases collected from the oxidized sludge tank and vacuum filters. This unit proved inadequate and was replaced with a high temperature gas fired afterburner which receives the odorous gas from the oxidized sludge tank and vacuum filters. The original catalytic burner was retained for treatment of the decant tank off-gases and was equipped with an air scrubber using effluent water. The oxidized sludge tank and vacuum filters are fully enclosed. These modifications were effective in controlling plant odors.

The city of Bedford Heights, Ohio operated a Zimpro low pressure oxidation heat conditioning system at its 13,600-m^3/d (3.6-mgd) activated sludge plant from 1970 until 1975 when the heat treatment system was temporarily taken out of service (49). The sludge recycle streams caused odors when returned to the main stream liquid processing units. Off-gas from the oxidized sludge tank was identified as a major source of odor. The originally installed catalytic incinerator was not effective in controlling odors in its present condition. The use of ozone, as well as acid and caustic scrubbers was also found to be ineffective. At the time of the shutdown, the city planned to replace the existing odor control system with a high temperature, 815°C (1,500°F) gas-fired afterburner. However, since the initial shutdown of the Zimpro units, the city increased the operational staff, and repaired leaks in the covered oxidized sludge tank. The original catalytic incinerator was repaired and was reported to be effective in controlling odors from the oxidized sludge tank (49).

The Rockland County, New York Plant is a 38,000-m^3/d (10-mgd) activated sludge plant that utilizes an intermediate 200°C (400°F) Zimpro oxidation sludge conditioning process for treatment of primary and secondary sludges (49). Control of odors from the oxidized sludge tank was a difficult problem during the first 4 years of operation. The district used several gas burners to control odors from the oxidized sludge tank, but experienced problems due to low incineration temperatures and installation problems. These problems were overcome with the installation of a new 815°C (1,500°F) (exhaust temperature), gas-fired incinerator equipped with heat exchangers to raise the incoming air temperature to 415°C (780°F) and to recover 55 percent of the exhaust heat.

The cities of Muskogee, Oklahoma; Gresham, Oregon; and Portland, Oregon utilize direct combustion at 649-760°C (1,200-1,400°F) for odor control of off-gases from thermal sludge conditioning units.

The above case histories and examples indicate that a major source of odorous air from thermal conditioning of sludge is the oxidized sludge and decant tanks. The most frequently used method to treat this air stream has been either catalytic or high temperature incineration. Based on a survey of odor treatment methods at 36 operating treatment facilities, the following conclusions were drawn (49).

1. The odorous off-gases can be successfully treated by incineration at temperatures of 815°C (1,500°F).
2. Low temperature catalytic incineration has not been as successful as high temperature incineration.

3. Conversion of low temperature catalytic combustion processes to high temperature incineration has not been successful because of inadequate gas capacity and overheating of the stack.
4. The fuel requirements for high temperature incineration are costly, and use of effective heat exchange systems should be considered.

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National Technical Information Service
5285 Port Royal Road
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(703) 487-4650

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Chapter 5

Designing to Avoid Odor and Corrosion in New Wastewater Collection Systems

5.1 Introduction

The potential for sulfide and odor generation in new sewer systems must be fully evaluated in the design stage based on the characteristics and properties of odor-causing compounds and the principles of control described in Chapters 2 and 3 of this manual.

Warm temperatures, flat topography and large sewer service areas are physical conditions which, unless specifically considered during design, are likely to result in sulfide odors and sulfide-induced corrosion within the collection systems. Proper selection of slopes, rational design of hydraulic structures such as drops and junctions, proper design of pumping stations, wet wells, holding basins, and force mains along with the provision of adequate ventilation are all critical elements of a total sewer system design that are necessary to minimize sulfide generation potential.

Although designing to avoid sulfide generation may increase the capital cost of a new sewer system, this approach is technically and economically preferable to having to control sulfides after they become a problem.

This chapter provides guidance for eliminating or minimizing the generation of sulfides in the design of new wastewater collection systems. Specific reference is made to control of H_2S, since this is the most prevalent odor source associated with wastewater conveyance systems.

Design procedures outlined for sulfide control will often be applicable for control of other odor-producing compounds present in municipal wastewater, since many of the design concepts presented deal with preventing the anaerobic conditions under which undesirable odors are more likely to occur.

5.2 Hydraulic Design

5.2.1 Slope

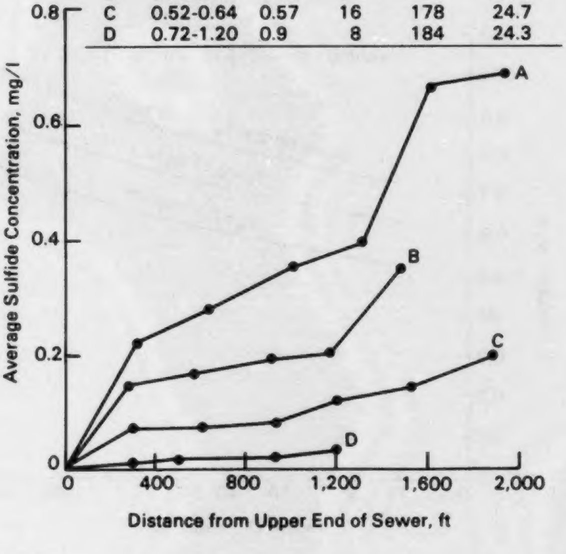
Slope is the key criterion in designing a wastewater collection system to avoid sulfide problems. Sewers designed with long runs at minimum slope are prone to sulfide generation due to long residence times, poor oxygen transfer, and deposition of solids. Sulfide generation can be a serious problem in new sewers, where actual flows are much less than design flows during the early lifetime of the system, and velocities are inadequate to maintain solids in suspension.

In the 1950's, a study was made of small collecting sewers in southern California. The sewers, all 15 and 20 cm (6 and 8 in) in diameter, were divided into four slope classes. The results from this study are shown in Figure 5-1 (1). This figure clearly shows the effect of slope and sewer length on sulfide generation. Steeper slopes increase turbulence and oxygen transfer, thus maintaining aerobic conditions in the wastewater and preventing significant sulfide generation. Although the values shown in Figure 5-1 for average sulfide concentration appear relatively low, peak sulfide concentrations were as much as four times the average (1).

In designing a wastewater collection system to minimize sulfide generation, velocities should be

Figure 5-1. **Sulfide occurrence in small sewers (1).**

Line	Slope (%) Range	Slope (%) Avg.	No. of Sewers	Avg. Results BOD	Avg. Results Temp. (°C)
A	0.20-0.25	0.23	18	253	24.7
B	0.32-0.46	0.40	22	212	24.4
C	0.52-0.64	0.57	16	178	24.7
D	0.72-1.20	0.9	8	184	24.3



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sufficient to prevent deposition of solids. Current conventional design practice recommends that a minimum velocity of 0.6 m/s (2 ft/s) be achieved regardless of pipe size to maintain a self-cleaning action in sewers. Another approach is to maintain a minimum boundary shear stress to prevent suspended particles from settling out on the invert.

The minimum horizontal velocity required to suspend particles of known characteristics can be computed using the following equation (2):

$$V_H = \frac{8k(s-1)gd^{1/2}}{f} \qquad (5\text{-}1)$$

where,

V_H = horizontal velocity that will just produce scour, m/s

k = constant which depends on type of material being scoured (typically 0.04 to 0.06)

s = specific gravity of particles

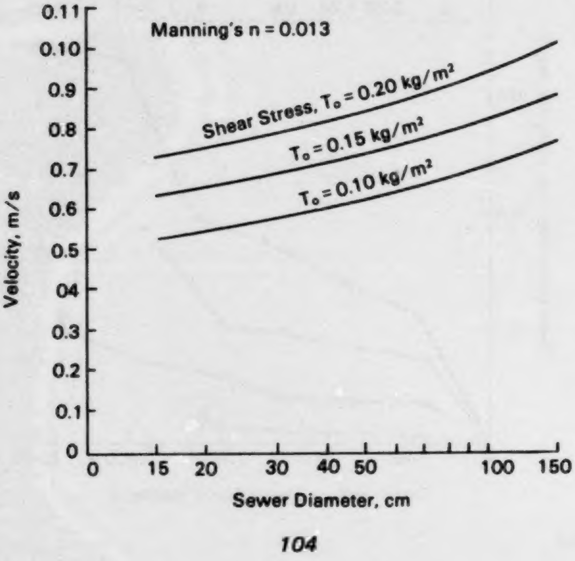
g = acceleration due to gravity = 9.8 m/s^2

d = diameter of particles, m

f = Darcy-Weisbach friction factor (typically 0.02 to 0.03)

If required minimum velocity is established based on maintaining a constant boundary shear stress, minimum velocities deviate from the recommended 0.6 m/s (2 ft/s) as a function of pipe size. Figure 5-2 shows minimum velocities required to maintain a constant shear stress as a function of pipe size (3). If a boundary shear stress of T_o = 0.15 kg/m^2 (0.03 lb/ft^2) is used, the minimum velocity requirement exceeds 0.6 m/s (2 ft/s) at pipe diameters greater than 35 cm (14 in), but is less than 0.6 m/s (2 ft/s) at smaller pipe diameters. This suggests that larger pipes require greater slopes to maintain adequate scouring velocities. For sewers with Manning's n = 0.013 or less, a design boundary shear stress in the range of 0.15 to 0.20 kg/m^2 (0.03-0.04 lb/ft^2) will likely keep self-cleaning sewer systems free from sulfide problems. For sewers with n = 0.015 or greater, a design shear stress of 0.2 kg/m^2 (0.04 lb/ft^2) should be used (3).

Figure 5-2. **Minimum scour velocity based on boundary shear stress (3).**



It should be noted that the often recommended 0.6 m/s (2 ft/s) is a minimum velocity. It is desirable to have a velocity of 0.9 m/s (3 ft/s) or more whenever practical (2).

Pomeroy has developed guidance regarding flow-slope relationships for preventing sulfide buildup. This is shown as Figure 5-3 (1). The curves are based on an assumed effective BOD (EBOD) of 500 mg/l. Here, EBOD represents the BOD during the daily maximum 6-hour flow period during the three hottest months of the year. The calculation of EBOD for Figure 5-3 is:

$$\text{EBOD} = 1.25\ \text{BOD}_5 \times 1.07^{(T-20)} \qquad (5\text{-}2)$$

where,

EBOD = effective BOD, mg/l

1.25 = factor to convert average daily BOD to maximum 6-hour flow BOD

BOD$_5$ = standard 5-day BOD, mg/l

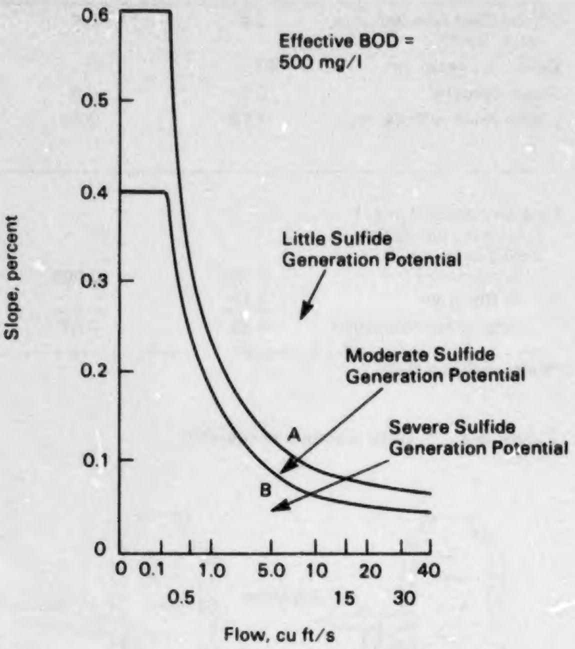
T = average wastewater temperature for three hottest months, °C

In the development of Figure 5-3, it is assumed that the depth of flow does not exceed two-thirds of the pipe diameter, and that the effective slope is calculated upstream of the point of interest over distances representing flow times of approximately 1 hour.

With an assumed EBOD of 500 mg/l, a system designed with a flow-slope relationship falling on or above Curve A may be expected to produce very little sulfide, rarely more than 0.1 to 0.2 mg/l of dissolved sulfide. Use of the flow-slope design points between Curves A and B may result in moderate sulfide concentrations which may cause odor and corrosion problems at points of high turbulence. Flow-slope design points falling below Curve B may result in substantial sulfide generation.

Figure 5-3 is intended only as a guide to predict when sulfide generation is likely to be a problem for certain slopes and flows, and is not to be used for detailed design purposes. Where wastewater characteristics vary from the assumed EBOD of 500 mg/l, the flow-

Figure 5-3. Flow—slope relationships as guides to sulfide forecasting (1).



slope relationships will increase or decrease in proportion to the square roots of the ratio of EBOD's. For a design EBOD other than 500 mg/l the minimum acceptable slope can be calculated according to (4):

$$S_D = \frac{EBOD_D^{1/2}}{500} \times S_c \qquad (5\text{-}3)$$

where,

S_D = minimum acceptable slope for design EBOD

S_c = minimum slope from Figure 5-3

$EBOD_D$ = design EBOD (mg/l)

Figure 5-3 is based on the average slope of sewer runs with lengths ranging from 365 to 580 m (1,200 to 1,900 ft). Even if the necessary average slope to prevent sulfide generation is attained, specific reaches should be checked to ensure that self-cleaning velocities are maintained where feasible, and that a minimum scouring velocity of 0.6 m/s (2 ft/s) is maintained during peak daily flow conditions. The recommended scouring velocity of 0.6 m/s (2 ft/s) for pipes flowing one-half full at design flow can result in velocities as low as 0.2 m/s (0.67 ft/s) during low flow periods early in the design lifetime of the system, thus allowing deposition of sewage solids. While this is undesirable, it cannot be economically avoided in certain instances. Sulfide generated from accumulated solids is generally much less critical than that generated from the slime layer, especially when the accumulated solids are flushed from the system on a daily basis.

Choice of a design slope depends on several factors other than flow and EBOD, including topography, subsurface conditions, depth of service laterals, pipe size and material, as well as overall economic trade-offs between gravity flow vs. pumped systems. If sewage pumping is required, a savings in pumping head by minimizing slopes should not, by itself, be a reason for using slopes that will result in significant sulfide generation. If choice of slopes for use in Figure 5-3 results in points falling on or below Curve B, indicating high potential for sulfide generation, Equation 2-25 should be used to calculate estimated sulfide buildup. Equation 2-28 can then be used to estimate the rate of corrosion of the pipe material. If the corrosion rate is such that the expected lifetime of the pipe is less than the design lifetime, several options are available to the engineer. These include use of steeper slopes or other means to promote natural reaeration, injection of air or oxygen, addition of chemicals, or selection of materials that are more resistant to corrosion. In general, the last option would be least desirable since, although rates of corrosion of pipe materials may be reduced, sulfide levels may still be high and may result in substantial odor generation.

Design lifetime is an important parameter in considering sulfide generation and subsequent corrosion of pipe materials in wastewater collection systems. EPA cost-effectiveness guidelines recommend a useful life of 50 years for wastewater conveyance structures, including collection systems, outfall pipes, interceptors, force mains, and tunnels. For special situations, as with sewers designed for interim service, shorter design lifetimes may be selected that are consistent with the planning objectives of the municipality.

5.2.2 Pipe Size

If sulfide generation has been determined to be a potential problem, larger pipe sizes may be selected to improve the rate of reaeration. A larger pipe for the same flow rate and slope reduces the mean hydraulic depth (cross-sectional area of the stream divided by surface area), which increases surface area available for reaeration (Equation 2-14). Figure 2-9 shows that reducing the relative depth of flow from 0.75 to 0.5 approximately doubles the reaeration rate. Adequate scouring velocities must be maintained if larger pipe is used, but this is not normally a problem since for a given flow and slope, velocity is influenced very little by pipe size.

Force mains have often been constructed of minimum diameter pipe in order to reduce detention time and avoid sulfide buildup. However, the smaller pipe has a greater ratio of slime-supporting pipe wall to volume of wastewater, partially offsetting the benefit of reduced detention time. Choice of minimum pipe size is, therefore, not considered to be of significant value in reducing sulfide generation.

5.2.3 Drops and Falls

For wastewater containing little or no dissolved sulfide, drop structures can result in the wastewater stream picking up substantial amounts of oxygen, helping to maintain aerobic conditions and preventing sulfide generation. However, for wastewater containing dissolved sulfide, the turbulence associated with drops or falls will release H_2S from the stream, resulting in odors and corrosion.

The benefits of reaeration through drops and falls were discussed in Chapter 2. Figure 5-4 shows two alternatives for grading a sewer. Alternative A employs a lesser slope but allows free fall of the wastewater at manhole 2. Alternative B shows the more conventional approach of increasing the slope of the line between manholes 1 and 2. Table 5-1 was developed by Thistlethwayte to show the impact of the two alternative designs on oxygen absorption (5). Note that the expected oxygen absorption in the sewer using a drop of about 1.2 m (4 ft) is 50 times the oxygen absorption without the drop. If some DO were present in the influent to manhole 1, the ratio would remain the same, but the amounts of oxygen absorbed would be reduced in proportion to the actual oxygen saturation deficits. Whether or not such a drop could be justified would depend on the DO levels upstream and the desired DO level downstream. Considering a DO increase of more than 3 mg/l by the use of a drop, it may be possible to lay the downstream sections at a flatter grade without exhausting the DO added by the drop (5).

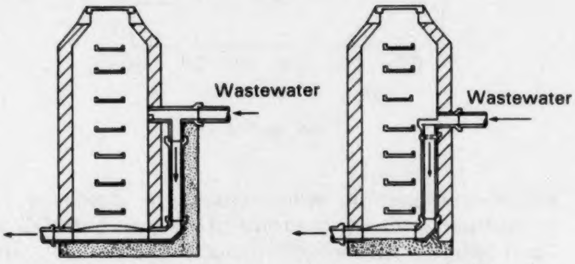
Typical drop manhole designs are shown in Figure 5-5 (6). Drops may be subject to clogging or stoppages due to bridging of sticks or other debris over the drop pipe. An alternate to the standard design places the drop inside the manhole, allowing easy access to the drop pipe. To avoid stoppages, the drop pipe may be of a larger diameter.

Table 5-1. **Oxygen Absorption in a Sewer With and Without a Drop (5)**

	Alternative A*	Alternative B*
Dry weather flow, m^3/min (max. 6 hr)	3.4	3.4
Sewer diameter, cm	61	61
Slope, percent	0.1	0.5
Wastewater velocity, m/s	0.53	0.96
Oxygen absorbed, mg/l (assuming saturation deficit of 10 mg/l)		
In the sewer	0.038	0.069
At the drop	3.18	-
Total oxygen absorbed	3.22	0.07

*Refer to Figure 5-4.

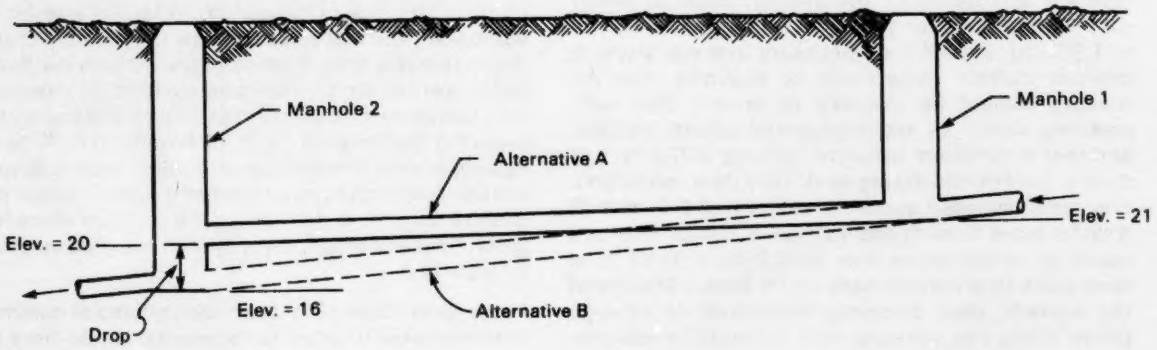
Figure 5-5. **Drop manhole designs (6).**



a) Drop Located Outside Manhold

b) Drop Located Inside Manhold for Easier Access

Figure 5-4. **Alternative sewer grading designs (5).**



For large flows and relatively large drop distances, vortex drops are sometimes used. In a vortex drop, the flow is directed tangentially to produce a spiral flow pattern. Advantages of vortex drops include: 1) maintenance of a continuous air core down the shaft; 2) excellent conditions for oxygen uptake; 3) no accumulation of solids or scum; 4) less likelihood of stoppages; and 5) better energy dissipation (5).

A diagram of a hydraulic fall is presented in Figure 3-17. Such designs are unlikely to have stoppages associated with sticks and debris, yet will provide substantial reaeration of the wastewater.

Drop manholes generally have not been used where it is economically feasible to steepen the sewer because of potential maintenance problems and increased construction costs (6). However, where sulfide generation potential exists, well designed drops and falls are effective techniques for maintaining aerobic conditions and preventing sulfide generation.

Drops or falls are generally not recommended when appreciable amounts of dissolved sulfide are present in the wastewater. Turbulence will release sulfide from the stream, generating odors and potentially deteriorating the structure. If drops must be used under such conditions, construction materials must be selected based on anticipated corrosion problems. To avoid odors and downstream corrosion, mechanical ventilation should be used to move air from downstream sections back to the drop structure. Sewer ventilation is discussed in Section 5.3.

5.2.4 Junctions and Transitions

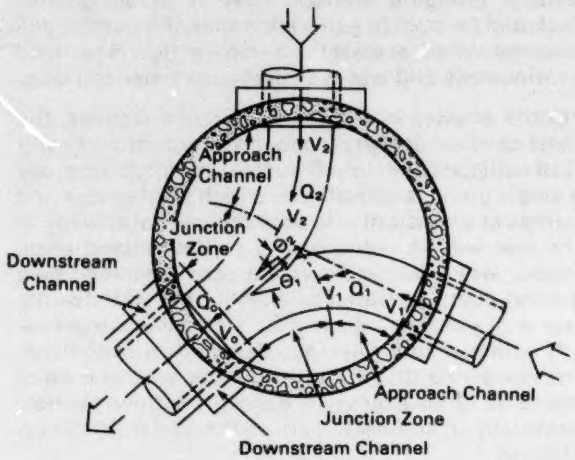
Sewer line junctions and transitions require special consideration because they offer an opportunity for both solids deposition and the release of dissolved sulfide. For aerobic wastewater, the major goal of junction design is to provide smooth transitions with minimum turbulence between incoming and outgoing lines so as to prevent eddy currents or low velocity points that will permit deposition of solids.

Design of junctions is more critical for sewers conveying septic wastewater, and special precautions must be taken to streamline the junction to minimize turbulence. Major factors that create turbulent conditions are:

1. Abrupt changes in grade between upstream and downstream sewer lines.
2. Large differences in velocity between two or more upstream sewer lines entering the same manhole.
3. Acute angles between upstream and downstream lines.
4. Large changes in flow between two or more upstream sewer lines that may be caused by upstream pumping or daily flow variation between different sewer service areas.
5. Large differences in flow between a trunk line sewer and tributary collector sewers.

A typical streamlined junction is shown in Figure 5-6.

Figure 5-6. Streamlined junction (5).



For one or more of the preceding conditions, turbulence will be minimized when the energy loss through the transition is minimum. This will be achieved when the following conditions are met (5):

1. The angle of convergence of the channels within the junction zone (θ_1 and θ_2) is as small as possible.
2. The channels are constructed so that the lateral momentum ($QV_1 \sin \theta_1$ and $Q_2V_2 \sin \theta_2$) of each of the incoming lines is reduced by the channel geometry before convergence of the two streams.
3. Velocity changes at the junction occur gradually.

5.2.5 Pumping Stations

The design of pumping stations is a critical element of sanitary sewer collection systems. Ideally, pumping stations should be designed so as not to increase the total sulfide generation potential of the collection system. This is often difficult, however, since contemporary design practice for pumping stations requires some wet-well storage of wastewater plus retention in the force main. When supplementary aeration is not provided, both of the above conditions will tend to increase the potential for sulfide generation by increasing the total residence time in the system, and by increasing the contact time of the wastewater with sulfide-generating slimes within the force main and the wet-well surfaces. Potential also exists for sulfide generation from solids deposition in the wet well if the wet-well design does not contain adequate bottom slopes and suction piping arrangements for their continuous removal.

Pumping stations may be generally classified as continuous or intermittent, depending chiefly on the size of the tributary sewer system and average and maximum design flow rates. Of the two, the intermittent pumping stations have a much greater potential for sulfide generation than the continuous stations, where at least the minimum flow is pumped continuously and wet-well detention times are less.

For the smaller intermittently pumped stations, the most common design practice is the provision of a wet well equipped with on-off pumping controls whereby a single pump is activated by a high level switch and pumps at a constant rate until the level of sewage in the wet well is reduced to a predetermined level. Higher wet weather flows are accommodated by a second pump activated by a level control. Since the wet well storage and pumping schedules are generally established for average design flow conditions, the residence time in both the wet well and force mains is often excessive during low flow periods, especially in the early part of the system's design lifetime.

The volume of the wet-well storage provided depends on the peak and average flows and the minimum duty cycle of the pumping system. Many of the smaller pumping applications within a size range of 380 to 11,350 m^3/d (0.1 to 3.0 mgd) utilize package pumping stations. Two pumps are normally provided, with a single pump sized to accommodate peak flow conditions. Duty cycles (time between successive starts) are typically 15 to 20 minutes, with minimum pumping times of 2 to 5 minutes. This design approach results in effective wet-well detention times of 5 to 15 minutes and total detention times of 7 to 20 minutes under average flow conditions. This may lead to excessive detention times and possible sulfide generation during low flow periods. Pomeroy indicates that significant sulfide generation will not occur in wet wells with detention times of less than 2 hours (1).

Wet wells should be as small as possible to minimize the potential for sulfide generation. A maximum wet-well design detention time of 30 minutes or less for all but the larger pump stations is recommended. The wet well should further be designed to avoid the accumulation of solids.

Wet-well detention times in larger pump stations equipped with variable speed pumps or with a combination of constant speed and variable speed pumps are generally sufficiently short to avoid sulfide generation.

Pump station wet wells are often the site of sulfide release when upstream DO levels are inadequate. Alternatives for sulfide control in pump stations are: 1) wet-well aeration; 2) chemical addition; 3) collection and treatment of H_2S-contaminated air; 4) air bypassing to a downstream section of sewer; and 5) injection of air or oxygen upstream in the force main. Wet-well aeration is effective in oxidizing dissolved sulfides, but can cause release of H_2S by air stripping. Short detention times in wet wells are insufficient to achieve sulfide oxidation, while longer detention times ($>$1 hour) may be adequate for complete oxidation. Wet-well aeration has the added advantage of temporarily increasing DO levels to prevent or reduce sulfide generation in downstream force mains. Currently, package pump station manufacturers do not include wet-well aeration as a part of their standard design, but some provide a mixing valve from the discharge side of the pump to the wet well to provide increased mixing.

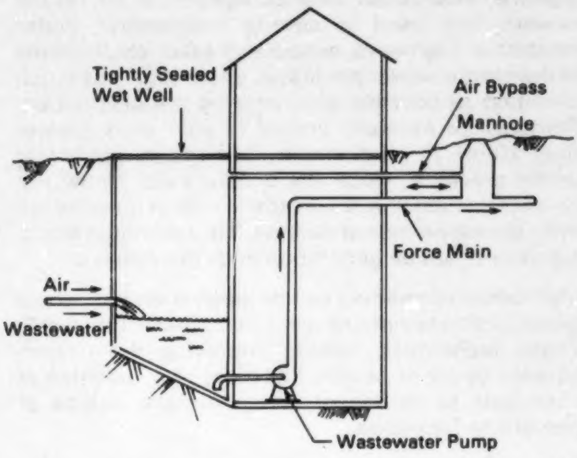
Wet-well aeration is a sulfide-control alternative that should be considered when excessive wet-well detention times must be provided and when the incoming wastewater exhibits DO, BOD, and ORP levels conducive to sulfide generation. This method is not recommended where significant sulfide ($>$0.5 mg/l) is present in the incoming wastewater. An ORP level of +100 millivolts has been used as the minimum design ORP for pump stations in a system design for Honolulu, Hawaii, to prevent sulfide generation in the downstream portions of the system (7). The target ORP level is dependent on the individual wastewater characteristics and the downstream collection system network.

All lift station designs should include an evaluation of the influent wastewater conditions, and of the impact of wet-well storage and force main sulfide-generation potential on downstream segments. In many cases, air or oxygen injection into the force main should be considered as an alternative to wet-well aeration. This method eliminates the problem of H_2S release, has the flexibility of providing the increased oxygenation capability where it is needed, and offers a higher and simpler level of control. Air and oxygen injection for sulfide control in collection systems is discussed in detail in Chapter 3.

The design of pump stations must also include an analysis of the sulfide generation potential of force mains. Sulfide generation within the force main is related to the wastewater characteristics, including DO present, EBOD, temperature, and sulfate concentration. Thistlethwayte has postulated that the concentration of sulfide buildup in a force main is proportional to $(L/D)(BOD)(SO_4)$ for continuously pumped systems, and that buildup for intermittently pumped systems is proportional to $(L/D)(BOD)(SO_4)(1.64)(V_c/V_i)$ where L is the sewer length, D is the sewer diameter, V_c is the velocity of a continuously pumped system, and V_i is the average velocity for an intermittently pumped system (5). It should be noted that other researchers have indicated that sulfate concentration does not limit sulfide buildup except at very low concentrations. Based on the work of Thistlethwayte, it was found that the total mass of sulfide generation was approximately equal in a given time period for a particular wastewater regardless of pumping cycles, but that the concentration of sulfide generated by intermittent pumping for certain pumping cycles could be several times that of a continuously pumped system. This finding is critical for downstream situations where the intermittent discharge of sulfide from force mains could create significant localized odor and corrosion problems. Since sulfide generation within force mains is due primarily to surface slime, larger force main sizes reduce the sulfide generation potential for a given design flow and wastewater characteristics, since they result in a smaller surface-area to cross-sectional-area ratio. The selection of force main size is normally made based on a cost analysis of increased pumping cost vs. the capital cost of the force main. For situations in which sulfide generation potential exists within the force main, a larger size force main may be warranted to reduce sulfide generation and subsequent sulfide control costs. Since most force mains may sometimes operate under conditions that produce sulfide, the discharge should be designed to minimize turbulence. Some circumstances may require special ventilation, sealing or collection and treatment of the odorous air.

An air bypass can often be used to shunt air around a pump station to control odor release from the wet well. This involves the use of a sealed wet well as shown in Figure 5-7 (1). An air bypass line is constructed between the wet well and the closest upstream gravity manhole. In some cases, this distance may be so long as to make this approach impractical. It is suggested that the air bypass line be approximately two-thirds of the force main pipe diameter (1). This approach would be especially applicable when a downstream gravity flow segment has significant reaeration potential and the force main distances are short. When significant sulfide generation is anticipated, separate air collection and treatment may be warranted. Separate off-gas treatment is discussed in Chapter 4.

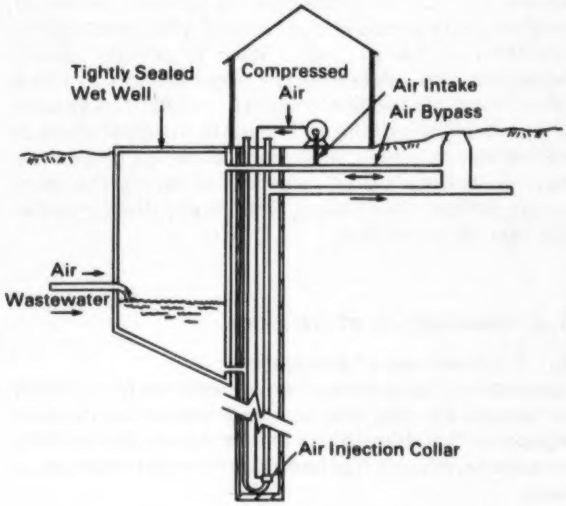
Figure 5-7. Pump station with an air bypass (1).



In cases where pumps are designed for lift only and the wastewater is not discharged into a pressure main, it may be desirable to consider use of an air lift pump. Air lift pumps are typically used only for low flow applications, where their ease of maintenance and reliable operation outweighs their low efficiency, which is limited to about 15 percent. However, economics of an air lift pump may improve substantially when considering that it serves two functions, pumping and aeration. Figure 5-8 shows a schematic diagram of a lift station employing an air lift pump with an air bypass line around the lift station (1). To eliminate a net discharge of air, the compressor takes air from the bypass connecting the sealed wet well and the downstream sewer.

The design of wet-well aeration systems, air or oxygen injection into force mains, and chemical addition is discussed in Chapter 2.

Figure 5-8. Air-lift pump station (1).



5.2.6 Siphons

Siphons, also called inverted siphons or depressed sewers, are used to convey wastewater under streams or highways, conduits or other obstructions to the normal sewer grade line, and to regain as much elevation as possible after passing the obstruction. Siphons are normally limited to pipe sizes greater than 20-cm (8-in) diameter. Sewage in siphons is under pressure, since the conduits are below the hydraulic grade line. Because the siphon remains full even during periods of no flow, it is a potential site of significant sulfide generation and odor release.

Methods of controlling sulfide generation that would be applicable to siphons were discussed in Chapter 3. These techniques include improving the oxygen balance by air or oxygen injection, or by addition of chemicals to oxidize or precipitate the sulfide or prevent its formation.

Siphon design must consider the potential for odor release. Positive pressure develops in the atmosphere upstream of the siphon due to the downstream movement of air induced by the wastewater flow. Air thus tends to exhaust from the manhole at the siphon inlet and may escape in large amounts from small openings, such as pick holes in manhole covers. At less than maximum flow, wastewater dropping into the inlet may cause turbulence and odor release.

One technique that has been successfully used to minimize odor release at siphons is the use of air jumpers. These are pipes that take the air off the top of the inlet structure and convey it to the end of the siphon. In most cases, air jumpers run parallel to the siphon, although the pipe can be suspended above the hydraulic grade line. Provisions should be made to drain the air jumper to periodically remove accumulated condensate. Usually, the diameter of the air jumper pipe is approximately one-half that of the siphon (6). Solids deposition is another potential problem and siphons should normally be designed for velocities of 0.9 m/s (3.0 ft/s) to prevent solids deposition and subsequent odor generation. In some cases, multiple siphon lines are installed to ensure adequate velocities during the early design lifetime of the system. In these instances, the unused sewer line may be used as the air jumper line during the early design period when flow is small and sulfide generation may be a problem.

5.3 Ventilation of Sewers

5.3.1 Objectives of Ventilation

Ventilation of sewers is often undertaken for a variety of reasons. For the most part, only the control of odors is practically achievable with ventilation. Some of the reasons ventilation has been attempted are discussed here.

5.3.1.1 Increasing the Oxygen Content of the Sewer Atmosphere

The oxygen content of the sewer atmosphere does not change significantly as a result of the septicity of the wastewater. In partially filled sewers, rise and fall of the liquid level results in displacement and replacement of air, and there is normally a downstream flow of air due to a drag effect between the air-sewage interface. Oxygen concentrations in such sewers are rarely less than 90 percent of normal. If oxygen concentrations are above 90 percent of normal, ventilation is unlikely to make a significant difference in the oxygen balance of the stream.

5.3.1.2 Reducing the Atmospheric H_2S Concentrations

Although it would seem feasible to ventilate sewers to reduce the atmospheric sulfide concentrations and thus control corrosion, this approach has little practical value. In order to have measureable results, complete replacement of the sewer atmosphere with fresh air would be required at frequent intervals. Even if this approach were economical, there would be the problem of disposal of large volumes of malodorous air.

5.3.1.3 Drying the Walls of Sewers and Other Structures

The oxidation of hydrogen sulfide gas to sulfuric acid does not occur if the surface is dry, since moisture must be present for bacterial oxidation of H_2S. Ventilation has been used with the objective of drying sewer walls. Thistlethwayte estimated that when the relative humidity of the sewer atmosphere exceeds 80 to 85 percent, sufficient moisture will be present on the walls to support bacterial activity (5). Thistlethwayte also proposes a design procedure for ventilation of sewers to control humidity, but indicates that in most cases this approach is not practicable. This is due to the rapid increase in relative humidity of ventilation air with flow along the sewer, the large number of ventilation stations required, and the significant increase in operation and maintenance costs. Pomeroy also indicates that this approach is impractical for year-round protection for even typical sewer distances between manholes (1).

5.3.1.4 Preventing Lethal Atmospheres

Portable fans or blowers are often used to ventilate manholes before workers enter. This is acceptable practice for localized conditions, provided other normal safety procedures are followed. However, it is questionable as to whether this practice would provide a safe environment between manholes. It is not feasible to ventilate large sections of a sewer system sufficiently to assure a safe environment for sewer workers.

5.3.1.5 Preventing Explosive Atmospheres

Explosions in sewers generally result from the presence of large amounts of volatile hydrocarbons or from leaking natural gas mains. Only under very unusual conditions do explosions result from accumulation of sewer gases. Because of the unpredictable causes of explosions and the conditions under which they occur, it is unlikely that ventilation could assure protection from explosions in a wastewater collection system.

5.3.1.6 Controlling Odor Emissions

Sewer ventilation can withdraw malodorous air at one point in order to prevent odor emissions at other locations. Normally, contaminated air must undergo treatment by one or more of the techniques discussed in Chapter 4. Ventilation is often practiced at wastewater treatment plants, where air is withdrawn at the downstream terminus of the sewer (plant headworks) and either treated separately or piped to existing biological stabilization processes for removal of odors. Although most other possible objectives have not been achieved by practical levels of ventilation alone, control of odor emissions can be effectively served by continuous ventilation.

5.3.2 Methods of Ventilation

Ventilation of a sewer can occur through both natural and mechanical means. Virtually all sewers incorporate some method of natural ventilation. Mechanical ventilation, on the other hand, is normally employed only in response to complaints of odor emissions from a portion of the collection system following the original design. The two methods are discussed below.

5.3.2.1 Natural Ventilation

Collection systems in the United States do not normally incorporate special vents or hardware to assist in natural sewer ventilation. Rather, manholes and building vents are generally considered adequate to keep sewers sufficiently ventilated (6).

Natural ventilation occurs from the following forces (5)(8).

1. Change in barometric pressure along the sewer
2. Wind velocities past vents
3. Frictional drag of wastewater on sewer air
4. Rise and fall of the wastewater level in the sewer
5. Relative density differences of sewer air and outside air

The degree of natural ventilation which occurs in a sewer is difficult to predict, since fluctuations in the above variables may change both the direction of movement and velocity of the air contained in the sewer.

Whereas no special provisions are normally made to enhance natural ventilation of sewers in the United States, special ventilation systems are routinely incorporated into sewer designs in the United Kingdom and Australia (5). The reason for this is that collection systems designed in the United Kingdom and Australia have typically incorporated "boundary traps" or "running traps" at building sewers or house laterals, which effectively prevent the transfer of air between the sewer and building vents. Since the building vent is no longer a source of ventilation air, induct and educt stacks are placed at various locations in the collection system to allow air movement into and out of the sewer. Research on natural sewer ventilation systems is discussed in References 7 and 8, and detailed design procedures for such ventilation systems are presented in Reference 4.

5.3.2.2 Mechanical Ventilation

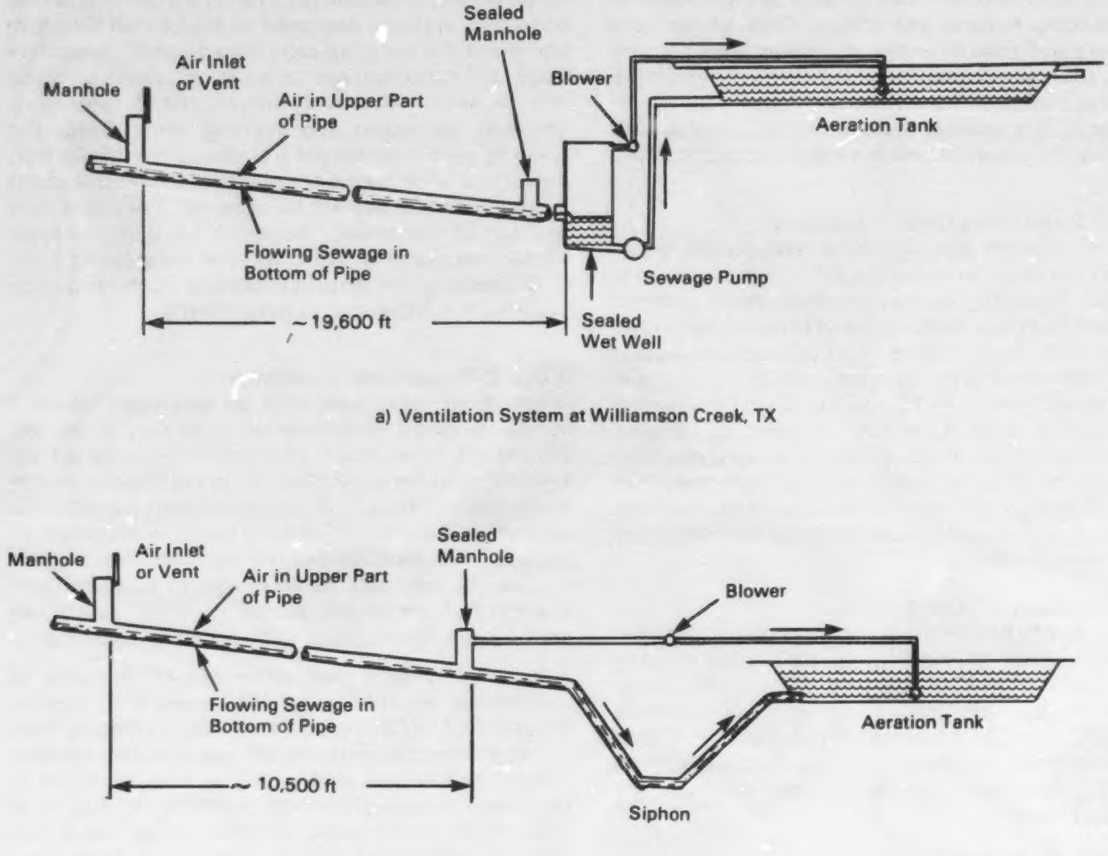
Mechanical ventilation may be employed where a constant velocity and direction of air flow is desired. This may be necessary where odor emissions from sewers must be controlled, as in residential neighborhoods, or where hydraulic conditions that occur in siphons or surcharged sewers result in stagnant air pockets with reduced oxygen contents. Mechanical ventilation may also be employed at headworks of wastewater treatment plants in order to convey malodorous sewer gases to odor control systems.

Figure 5-9 shows two examples of the use of mechanical ventilation for odor control in Austin, Texas (10). At Williamson Creek, odors escaping from septic wastewater entering the wet well necessitated sealing of the wet well and upstream manhole to allow withdrawal of air from 5,980 m (19,600 ft) of 106-cm (42-in) diameter concrete outfall line. A 7.1-m^3/min (250-scfm) blower was used to remove odorous gases from the sewer and discharge them to an aerated stabilization pond. This approach was successful in controlling odors from the system.

A similar approach was used for the Walnut Creek system. This was a total gravity system which included a siphon for conveying wastewater under Walnut Creek. Two 14.2-m^3/min (500-scfm) blowers were used to remove odorous gases from 3,200 m (10,500 ft) of concrete sewer at a sealed manhole upstream of the siphon. The blower discharged the gases through air lift pumps in the aeration basin of the treatment plant to achieve better mixing of the tank contents and absorption and oxidation of the odorous components of the gas in the aerated liquid.

Ventilation of pumping stations is part of normal design procedures for these structures. A minimum of 12 air changes per hour is recommended for continuously ventilated wet wells and 30 air changes per hour for intermittently ventilated wet wells. A minimum of 6 air changes per hour is recommended for continuously ventilated dry wells and 30 air

Figure 5-9. Forced draft ventilation for odor control, Austin, TX (9).


Sealed Manhole
Air Inlet or Vent
Manhole
Blower
Air in Upper Part of Pipe
Aeration Tank
Flowing Sewage in Bottom of Pipe
Sewage Pump
~19,600 ft
Sealed Wet Well
Sealed Manhole
Manhole
Air Inlet or Vent
Air in Upper Part of Pipe
Blower
Flowing Sewage in Bottom of Pipe
Aeration Tank
~ 10,500 ft
Siphon

a) Ventilation System at Williamson Creek, TX

b) Ventilation System at Walnut Creek, TX

changes per hour for intermittently ventilated dry wells and other below grade structures (11).

5.4 Selection of Materials

Materials selection is a critical aspect in design of wastewater collection systems in which sulfide generation is likely to pose problems. The additional expense of using materials with greater degree of corrosion resistance may be justified by the cost savings for replacement or rehabilitation of deteriorated structures at some later date. The following discussion describes the various materials used in collection systems, with particular emphasis on the corrosion-resistant properties of each material.

5.4.1 Pipe Materials

If sulfide is expected to be present in sufficient quantities to cause corrosion, consideration must be given to the use of pipe materials with higher degrees of corrosion resistance. Design considerations in selecting such materials are (1):

1. Availability of the materials in the pipe sizes required
2. Minimum and maximum levels of sulfide expected in the wastewater
3. Factors other than acid resistance of the pipe (abrasion resistance, stress-corrosion resistance, load-bearing strength, and other durability considerations)
4. Hydraulic characteristics of the materials under conditions of actual use
5. Other advantages or disadvantages of the material (ease of installation, resistance to infiltration, flexibility, etc.)
6. Expected future service requirements
7. Relative costs vs. expected service lifetimes of various kinds of pipe

5.4.1.1 Concrete Pipe

Concrete is one of the most common materials used in construction of sewer pipe, and is virtually the only material used for large diameter trunk sewers.

Several alternatives are available to extend the design lifetime of concrete pipe in corrosive atmospheres found in sewers. These include: 1) specification of calcareous aggregate, which increases the overall alkalinity of the concrete; 2) specification of additional wall thickness to serve as sacrificial material; and 3) use of liners or coatings with high degrees of corrosion resistance on the interior pipe walls.

Alkalinity of the concrete pipe and the thickness of concrete cover over the reinforcing steel have been used in the development of the "life factor" equation (12):

$$(A)(z) = 0.45\,k\,\phi_{sw}\,L \qquad (5\text{-}4)$$

where,

$(A)(z)$ = life factor = product of alkalinity [expressed as a weight fraction; i.e., alkalinity as ($CaCO_3$) as fraction of cured pipe weight] and thickness of allowable concrete loss (in)

k = coefficient of efficiency for acid reaction (see Equation 2-30)

ϕ_{sw} = flux of H_2S to the pipe wall, $g/m^2/hr$ (see Equation 2-20)

L = desired design lifetime, years

This equation is useful in that, if the H_2S flux is calculated based on assumed conditions, the desired service life of the pipe can be entered and a life factor computed, which allows flexibility in selecting various combinations of pipe thickness and alkalinity of the concrete.

Example:

Assume $L = 50$ years

$\phi_{sw} = 0.03\ g/m^2/hr$

$k = 0.7$

$$(A)(z) = 0.45\,k\,\phi_{sw}\,L = (0.45)(0.7)(0.03)(50) = 0.47$$

This life factor could theoretically be met by numerous combinations of alkalinity and wall thickness, examples of which are shown below.

Aggregate	Alkalinity of Concrete(A)	Concrete Cover (z)	(A)(z)
	weight fraction	in	
Granitic	0.2	2.4	0.48
50-percent Calcareous	0.50	1.0	0.50
100-percent Calcareous	0.85	0.6	0.51

A manufacturer can thus meet the required life factor by using the combination of alkalinity and wall thickness that is most economical and suitable to the expected use and to the production process.

The alkalinity of concrete varies with the cement content and type of aggregate. Ranges of alkalinity (weight fraction) for concrete pipe containing 352 kg cement/m^3 (594 lb/cu yd) are shown below for various aggregates:

Granitic aggregate:	Alkalinity = 0.18 to 0.22
50-percent calcareous aggregate:	Alkalinity = 0.4 to 0.6
100-percent calcareous aggregate:	Alkalinity = 0.8 to 0.9

Procedures for obtaining interior wall cores of concrete pipe and for determining alkalinity of the samples are described in References 13 and 14.

Alkalinity of the interior wall of concrete pipe will also vary with the method of manufacturing. Centrifugally spun pipe generally has a higher interior wall alkalinity than cast pipe due to the migration of cement toward the interior wall during production (14).

It should be noted that not all concrete pipe manufacturers have ready access to calcareous aggregates. Most manufacturers should be able to meet life factor, (A)(z) design specifications through a combination of concrete alkalinity and wall thickness.

5.4.1.2 Asbestos Cement Pipe

Asbestos cement pipe is subject to attack by sulfuric acid. Because the cement content is higher than for reinforced concrete pipe, the alkalinity may also be higher, depending on the type of aggregate used. However, corrosion of asbestos cement pipe immediately begins to degrade the structural section of the pipe, as opposed to corrosion of reinforced concrete pipe in which the concrete cover over the reinforcing steel is degraded before the structural integrity of the pipe is affected.

Although variability in the alkalinity of asbestos cement pipe is limited, the life factor design approach can be employed to determine required thickness. Alkalinity of asbestos cement pipe is typically in the range of 0.5 to 0.6 (12).

It should be noted that asbestos cement pipe is banned in many areas, and is generally not available in the United States because of the known health effects of asbestos fibers.

5.4.1.3 Vitrified Clay Pipe

Vitrified clay pipe is immune to attack by sulfuric acid and most volatile industrial waste products, and as such is a suitable material for use where high sulfide concentrations are expected. Elastomeric jointing materials, which are resistant to attack by many corrosive materials, should be used with the vitrified clay pipe. Properly laid and jointed, vitrified clay pipe will remain serviceable for a very long time if not disturbed by external forces. Vitrified clay pipe is available in sizes ranging from 10 cm (4 in) to 107 cm (42 in); the larger sizes may not be available in all parts of the country.

5.4.1.4 Reinforced Plastic Mortar Pipe

Reinforced plastic mortar pipe is constructed of polyester resin mixed with sand and reinforced with fiberglass. The resulting product is not subject to sulfuric acid attack unless the glass fibers are exposed due to damage during handling or deflections of the pipe. If the fibers are exposed, acid can creep along the fibers and react with impurities in the fibers. The oldest sewer constructed of reinforced plastic mortar pipe is a trunk sewer installed in San Jose, California in 1966 (1)(12).

5.4.1.5 Homogeneous Plastic Pipe

Polyvinyl chloride (PVC), acrylonitrile-butadiene-styrene (ABS), and polyethylene (PE) are pipe materials resistant to sulfuric acid attack and thus suitable for use where high sulfide concentrations are expected. Care must be given to bedding and backfill to keep pipe deflections to an acceptable minimum (1)(12). ABS pipe is very susceptible to stress corrosion.

5.4.1.6 Steel Pipe

Steel pipe is subject to corrosion by sulfuric acid as well as by H_2S in the presence of oxygen when the sewer is flowing partially full. The oxidation product, iron sulfide, can accumulate to an extent that the hydraulic capacity and the structural integrity of the line are significantly reduced.

5.4.1.7 Ductile Iron Pipe

Ductile iron pipe has replaced gray cast iron pipe for use in wastewater collection systems. Gray cast iron pressure pipe is no longer manufactured in the United States.

Ductile iron pipe normally lasts longer than steel pipe due to the increased wall thickness. However, iron is subject to corrosion in the presence of oxygen as is steel, and the bulky corrosion products may accumulate and restrict the cross-sectional area of a pipe and affect the structural integrity of the pipe.

When iron pipe is corroded by H_2S gas, sulfuric acid, or other agents, the process proceeds by graphitization, which involves dissolution and removal of the iron crystals, leaving behind non-metallic components such as graphite, carbides, silicides of iron, and corrosion products. Although the pipe may appear to be in good condition, its structural strength is often greatly reduced (15).

5.4.2 Pipe Linings and Protective Coatings

Many different types of linings and coatings have been used in attempts to protect pipe from corrosion due to wastewaters containing sulfides. Unfortunately, success with these materials has been quite variable. The problem is in achieving a sealed lining that is firmly affixed to the interior pipe wall and which has no defects, pinholes, or construction damage that would allow penetration of acid to the pipe. Such defects can result in localized corrosion occurring at a greater rate than if the acid attack were distributed over the total pipe surface. Acid, penetrating through pinhole-sized defects, attacks the underlying material, and the accumulation of expansive corrosion products eventually ruptures the lining or coating, allowing greater acid penetration and progressive deterioration of the pipe. Coatings can be painted, sprayed, or troweled onto the interior surface of the pipe, and linings may be applied as preformed sheets or panels during manufacture of the concrete pipe.

5.4.2.1 PVC Liners

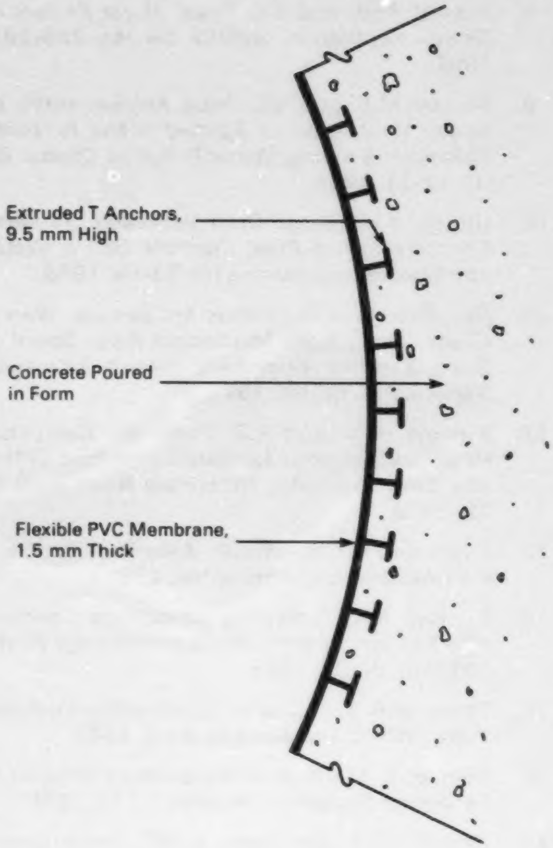
One of the few lining systems which has been used successfully for long-term protection of concrete is a PVC liner mechanically attached to the concrete. The liner consists of sheets of plasticized PVC approximately 1.5-mm (1/16-in) thick with T-shaped keys running longitudinally on one face. The sheets are fastened to the forms, key side in, before pouring of the concrete during manufacture of the pipe. In the finished pipe, the keys are firmly imbedded in the concrete. The PVC sheets are heat-welded at the pipe joints to produce a completely sealed liner. A schematic diagram of a T-lock PVC liner for concrete pipe is shown in Figure 5-10 (16).

Some problems have arisen in the installation of such liners in cast-in-place concrete pipe because of the difficulty in imbedding all the keys. No such problems have been reported for factory made pipe. Although PVC liners may be subject to damage from very high turbulence or from mechanical cleaning tools, proper design and operation can overcome such problems (1). In California, PVC sheet lining has been successfully used for concrete sewer pipe protection for over 30 years (17).

5.4.2.2 Vitrified Clay Liner Plates

Vitrified clay liner plates mechanically locked to concrete pipe have also been used. However, porosity of the clay allowed acid to diffuse into the concrete, softening and expanding it, which resulted in cracking of the plates and breaking of the lugs which lock the

Figure 5-10. **T-lock PVC liner for concrete pipe.**


Extruded T Anchors, 9.5 mm High
Concrete Poured in Form
Flexible PVC Membrane, 1.5 mm Thick

plates to the interior wall of the pipe (1)(18). This product is no longer available in many areas.

5.4.2.3 Thick Film Coal Tar/Epoxy Coatings

One of the few pipe coatings for which long-term performance data are available is the thick film coal tar/epoxy coating. Such coatings are spray-applied either during or after manufacture of the pipe, but prior to pipe installation. Although many coatings are applied in relatively thin films <2.5 mm (10 mil), thick film coal tar/epoxy coatings are generally applied with a minimum thickness of 10 mm (40 mil). In some cases, film thicknesses of up to 25 mm (100 mil) are specified. Coal tar/epoxy coatings are used for both metal and concrete pipe.

For successful long-term performance of thick film coal tar/epoxy coatings, the following conditions must be met:

1. Adequate surface preparation—sandblasting of the surface to remove all foreign materials and contaminants; removal of dust.
2. Adequate film thickness—minimum film thickness of 10 mm (40 mil).
3. Adequate quality assurance procedures, including:
   a. Checks on wet film as applied
   b. Checks on dry film thickness
   c. Low voltage holiday detection on 100 percent of barrel surface
   d. Hanging-weight adhesion tests

5.4.2.4 Cement Mortar Liners

Cement mortar is often used as a liner for iron or steel pipe in wastewater applications. For conditions in which sulfide-induced corrosion may present a problem, additional liner thickness and/or alkalinity of the cement may be specified. The life factor design approach can be used to achieve a desired lifetime of the cement mortar lining. Alkalinities of mortar used in lining of ferrous pipe are typically 0.4 to 0.5 (12).

5.4.2.5 Other Pipe Lining Coatings

Another alternative lining material is type 3 lb L stainless steel sheeting with a thickness of 0.5 to 0.6 cm (0.18 to 0.25 in). These sheets may be used where PVC sheet liners may be subject to mechanical damage.

Numerous coatings are available for sewer pipe. Some of the more common materials not previously discussed include asphaltic compounds, polyethylene, and polyurethane. Asphaltic compounds have not proved to be successful in sewers in which H_2S is present. Volatile materials present in the wastewater can dissolve the coating, and scratches, defects, or pinholes allow acid to migrate to the pipe. Long-term field experience with polymeric materials, such as polyethylene and polyurethane, is limited.

5.4.3 Construction Materials for Appurtenances

In designing a wastewater collection system to avoid sulfide problems, selection of pipe materials is of paramount concern. However, the design engineer must also consider selection of materials for sewer appurtenances such as manholes, transition structures, and drops.

If relatively high sulfide concentrations are expected in the wastewater, such appurtenances may promote turbulence and release of H_2S, which can result in H_2S gas and sulfuric acid attack on both pipe and appurtenances. An example of this type of occurrence is described below.

In Port St. Lucie, Florida, a 10-cm (4-in) PVC pressure pipe carrying septic tank effluent from approximately

200 homes discharged into the concrete manhole of a larger diameter gravity sewer conveying raw wastewater. The septic tank effluent contributed approximately 20 percent of the total flow, and contained dissolved sulfide concentrations in excess of 10 mg/l. After approximately 8 years of service, severe deterioration of both the concrete manhole and the cast iron manhole cover was observed. This was attributed to the turbulence at the junction of the two streams, which released H_2S gas to the sewer atmosphere. The concrete manhole was replaced with one fabricated from fiberglass, and a drop pipe was installed to reduce turbulence. Although no deterioraton of the fiberglass manhole has been observed, evidence of corrosion has been noted at the next concrete manhole downstream.[1]

It is, therefore, necessary to carefully consider materials selection for all components of a wastewater collection system, including manholes, junctions, and drops, in which the presence of H_2S gas and sulfuric acid poses a potential corrosion problem.

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When an NTIS number is cited in a reference, that reference is available from:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650

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Chapter 6

Designing to Avoid Odor and Corrosion in New Wastewater Treatment Facilities

6.1 Introduction

Current design practices for wastewater treatment facilities do not normally consider the potential for odor generation and corrosion and the factors required to control and minimize these problems. Failure to adequately consider odor and corrosion control during design has been due to: 1) the difficulties in predicting and quantifying sulfide generation; 2) an over reliance on accepted criteria for aeration, hydraulic, and ventilation design; and 3) the absence of specific design information and guidance for odor and corrosion evaluation and control. It must be recognized that some odors will occur at virtually all wastewater treatment plants and that some background odor level cannot be totally avoided. The objective of odor and corrosion evaluation and control in new treatment plant design is to prevent or minimize corrosion and the occurrence of nuisance odor levels to the surrounding community. An odor intensity of 176 odor units/m^3 (5 odor units/cu ft) at the boundary of the treatment facility has been found satisfactory to avoid nuisance complaints (1).

One of the more important considerations in new plant design is the analysis of present and future wastewater characteristics and flows as they affect the hydraulic design of plant components, basin detention times, loadings to biological processes, and the sludge generation potential of the facility. Both underloaded and overloaded unit processes have potential for odor generation during the design lifetime of the facility. Significant odor generation has been observed for underloaded facilities due to excessive detention times in wet wells and holding basins and solids deposition resulting from low channel velocities within the plant. The more common situation, however, is odor generation resulting from organic overloading, inadequate supply of air, improper ventilation, or simply the failure to recognize that certain unit processes may require the implementation of special odor control technology.

An important design consideration is plant siting. Designing for odor control is more critical if the plant is sited close to residences, major highways, commercial developments, or other populated areas. Odors generated from a plant sited in a relatively remote location may be considered acceptable unless the severity is such that it is objectionable to plant operators and maintenance personnel.

Certain unit processes in wastewater treatment plants have increased the potential for odor generation and require special consideration. Physical-chemical systems are often subject to severe odor and corrosion problems due to the absence of aeration and the subsequent low oxidation-reduction potential (ORP). Septage and sludge handling systems are common sources of objectionable odors. Sludge treatment operations, especially those employing thermal conditioning processes, may require special attention to ensure adequate odor control.

The type and severity of odors vary significantly with the plant size and type of treatment employed. For example, thermal sludge conditioning units in large plants generate odors that often cause complaints from adjacent neighborhoods; these units may be the dominant odor source from facilities where the process is employed. Odors from preliminary treatment processes in large plants are controlled by enclosing the components in a building. However, in small plants where facilities are not enclosed, odors generated from preliminary treatment facilities are dispersed into the atmosphere. Sludge drying beds and unaerated storage lagoons commonly used at small plants may also be odor sources, particularly if the sludge is not well stabilized. The design of a given facility must consider the relative magnitude of all potential odor sources and the control methods necessary to reduce odors to acceptable levels. The more common odor-generating unit processes in wastewater treatment facilities are presented in Section 6.2.

Corrosion potential must also be considered during design. Basic design considerations for minimizing corrosion in treatment plants are presented in this chapter. Where corrosive agents such as H_2S or acids are known to be present in the wastewater, special provisions are required for effective corrosion control.

Although it may be difficult to predict with great certainty odor and corrosion problems over the lifetime of a wastewater treatment plant, cognizance of potential problems and adherence to certain design procedures can minimize their occurrence.

6.2 Common Sites of Odor Generation

Table 6-1 lists the more common odor generating unit processes in municipal wastewater treatment plants, and ranks their potential for odor generation. In general, the greatest potential for odor generation is associated with preliminary treatment of the raw wastewater, and with the storage and treatment of sludge.

Table 6-1. Potential for Odor Generation from Common Unit Processes in a Wastewater Treatment Plant

	Odor Potential*
Liquid Stream Processes	
Flow Equalization	H
Preaeration	H
Screening	H
Grit Removal	H
Primary Clarification	H
Stabilization	
Suspended Growth	L
Fixed Film	M
Chemical	H
Secondary Clarification	L
Tertiary Filtration	L
Disinfection	L
Sidestream Returns	H
Sludge Stream Processes	
Thickening/Holding	H
Aerobic Digestion	M
Anaerobic Digestion	M
Thermal Conditioning	H
Storage Lagoons	H
Dewatering	
Vacuum Filter	H
Centrifuge	H
Belt Filter	H
Filter Press	H
Drying Beds	H
Composting	H
Septage Handling	H

*L = Low
M = Moderate
H = High

6.2.1 Liquid Stream Processes

Preliminary treatment works are potential sources of odor since they process raw wastewater which contains putrescible organics and debris. Raw wastewater may also be septic and contain dissolved sulfides and other odorous gases. Flow equalization basins are generally aerated to maintain solids in suspension and to prevent septicity. However, incoming dissolved sulfides or other odorous gases may be released by such aeration. Preaeration basins, bar screens, and aerated grit chambers may also induce release of odorous vapors from the wastewater. Storage and handling of raw screenings and organically coated grit can result in odor generation from putrefaction of the organic materials. In small plants, these materials may be allowed to accumulate and decompose over periods of several days or more before quantities are sufficient to warrant removal and disposal.

Primary clarifiers may generate odors if the influent wastewater is high in sulfides or if settled sludge residence times are sufficiently long to allow septic conditions to develop. In many cases, insufficient sludge removal frequencies may allow septic conditions to develop in the settled sludge. Turbulence associated with the fall of primary effluent over the weirs can release H_2S and other odorous gases to the surrounding atmosphere. Scum handling systems can be significant sources of objectionable odors if not regularly flushed and cleaned. It is recommended that the sludge withdrawal schedule for primary clarifiers be established to limit sludge residence times to less than 1 hour under average flow conditions. Scum handling equipment should be designed to be easily flushed, degreased, and disinfected.

Liquid stream biological treatment processes, such as aeration basins and trickling filters, are not normally sites of significant odor generation if properly designed and operated. In organic overloading, however, DO depletion and septic conditions can develop. These conditions can occur with organic overloading of trickling filters, high seasonal waste loads, and first stage overloading of rotating biological contactors. Failure to provide for adequate mixing in an aeration basin can result in deposition and accumulation of organic solids in "dead zones," which then may become septic and generate odors.

Secondary clarifiers normally do not generate odors, since the incoming liquid is aerobic. However, sludge withdrawal rates that provide for greater than 1.5- to 2-hour sludge residence times can allow septic conditions to develop in the settled sludge.

Return of sidestreams to the headworks or the liquid stream processes has high potential for odor generation, since sidestreams from sludge stabilization, conditioning, thickening, and dewatering operations often contain high concentrations of organic and odorous materials that have high oxygen uptake rates and can become septic very quickly. Return of these streams under turbulent conditions allows odorous gases to be released to the atmosphere.

Other potential sources of odors are unit processes that have been permanently or temporarily taken out of service. Failure to provide complete dewatering and thorough cleaning of unused tankage can lead to odorous putrefaction of remaining solids and organic slimes.

6.2.2 Sludge Handling Processes

Sludge handling processes are normally the major sources of odors at most municipal wastewater treatment facilities. Primary sludge, due to its raw, unstabilized state, has a highly offensive odor and a low ORP. Secondary sludges from activated sludge or trickling filter systems are generally less objectionable, but when stored in holding tanks or subjected to thickening processes with residence times in excess of 1 to 2 hours, they may generate odors due to the reduction in ORP and development of septic conditions. Sludge stabilization processes such as aerobic and anaerobic digestion, are not normally significant odor sources if properly designed and operated. However, overloaded aerobic digesters or improperly operated anaerobic digesters with uncontrolled release of gaseous products can be odorous. Thermal sludge conditioning systems produce offensive odors that require special attention. Sludge handling facilities downstream of thermal conditioning processes, such as decant tanks and blending tanks should be covered and separate treatment provided for the gaseous discharges. This is because of the greater release of odor compounds at higher temperatures and the odor-producing chemical reactions that take place during the conditioning process.

Sludge dewatering units are also potential sites for odor generation, since physical removal of water releases odorous gases. Since dewatering units are normally housed in buildings, odor concentrations inside these facilities can easily reach objectionable levels. Adequate ventilation is necessary to maintain odor concentrations at an acceptable level in such enclosed spaces; however, the exhaust air may also have to be treated in many instances to prevent release of objectionable odors to the atmosphere.

Sludge storage tanks and lagoons are other potential odor generators due to long residence times and development of septic conditions. Odor generation from lagoons is more severe during sludge withdrawal or lagoon loading due to increased turbulence. Where lagoons must be located in highly populated areas, designs incorporating an aerated top layer of water have been successful in oxidizing sulfides released from bottom sediments. Temporary odor masking may be used during sludge removal. The major design considerations for lagoons involve analysis of atmospheric conditions and siting, including prevailing wind direction, local weather patterns, and proximity to populated areas.

6.3 General Design Considerations for Avoiding Odor Generation and Release

6.3.1 Hydraulic Design

Proper hydraulic design is critical for minimizing the potential for odor generation in a wastewater treatment plant. In general, self-cleaning velocities should be achieved in all channels and interprocess piping to prevent solids deposition. Minimum self-cleaning velocities of 0.45 m/s (1.5 ft/sec) at minimum flow are recommended to prevent solids deposition.

Velocities through horizontal flow grit chambers are normally maintained at a reasonable constant level of about 0.2 m/s (0.75 ft/sec) through the use of Sutro or proportional weirs at the channel outlets. Alternatively, a parabolic channel, approximated by a trapezoid, can be used for velocity control (2). Velocities of less than 0.2 m/s (0.75 ft/sec) will cause deposition of organic solids with the grit and subsequent generation of objectionable odors. Aerated grit chambers will release sulfides, if present, due to turbulence and short hydraulic detention times. Severe corrosion has been noted for mechanically cleaned and housed aerated grit chambers that have not been properly ventilated.

Rectangular interprocess distribution channels should be constructed with fillets at the bottom corners to avoid solids accumulation in "dead" flow zones that are difficult to clean (3). Channels with trapezoidal cross-sections can also be constructed to ensure self-cleaning velocities throughout the plant's lifetime. A common approach is to use aerated channels to maintain solids in suspension and to provide DO. This ensures no deposition of solids over a wide range of anticipated flows. Aeration systems for such channels are recommended and should be sized to provide air at the rate of 0.2 to 0.5 m^3/min per lineal meter of channel (2-5 cfm/lineal foot) (2).

Turbulence should be avoided for any process streams which may be septic and contain malodorous gases such as H_2S, since this will tend to release these gases to the atmosphere. For example, process sidestreams returned to the headworks, or other liquid stream processes should be introduced below water surface, since hydraulic drops will result in rapid release of gases that can cause odors and induce corrosion of metal and concrete.

Flow equalization, preaeration, and activated sludge aeration basins must be designed to prevent solids deposition and creation of "dead zones." This is accomplished by proper selection of basin geometry, and by appropriate sizing and placement of aeration equipment. A general rule of thumb for achieving adequate mixing is to provide a minimum air flow of 0.3 m^3/min per meter of tank length (3 cfm/ft) (2). Triangular baffles or fillets should be provided in the bottom corners to avoid accumulation of solids.

For plastic media trickling filters, minimum wetting rates are normally specified to ensure that the entire surface area is wetted, and to prevent filter fly nuisances and odor generation. Depending on the specific media employed, recommended minimum wetting rates vary from 5.7 to 28.5 l/min/m^2 (0.14 to 0.7 gpm/ft^2). Provisions should be made for periodic flooding of the filter to control larvae and organic slimes.

All tanks and chambers should be designed to allow complete dewatering and flushing to prevent septic wastewater, slimes or sludge from remaining and creating odors when the units are taken out of service.

6.3.2 Aeration

Aeration is a widely used and effective method of preventing sulfide generation in wastewater treatment plants. The proper design of aeration systems requires an understanding of both the benefits of preventing sulfide formation as well as potential detrimental effects of release of sulfide that might be present in solution. The following summarizes the effects of aeration for various applications in the design of new treatment facilities:

1. Aeration of waste streams containing appreciable dissolved sulfide (> 0.5 mg/l) will cause sulfide release that may increase the H_2S concentration in enclosed air spaces to several hundred times the odor threshold.
2. Aeration of sulfide-containing wastes in short detention time basins (<30 min) will not reduce dissolved sulfide levels sufficiently to prevent release.
3. Aeration of sulfide-bearing wastes for long detention periods or in the presence of active biomass will reduce dissolved sulfides to the point that stripping is not significant.
4. Aeration of wastes containing no appreciable dissolved sulfide is recommended whenever increased DO is desired to prevent low ORP's and subsequent sulfide generation.
5. Aeration is recommended to prevent solids deposition and to maintain adequate DO levels throughout the plant.

For distribution channels, air should normally be supplied at the rate of 0.2 to 0.5 m^3/min per lineal meter (2 to 5 cfm/lineal foot). Preaeration basins should be designed to provide air at the rate of 0.7 to 3 m^3/m^3 (0.1 to 0.4 ft^3/gal) of wastewater, while flow equalization basins should be supplied with air at a minimum rate of 9.4 m^3/min/1,000 m^3(1.25 cfm/1,000 gal) of storage capacity (2)(3).

6.3.3 Covering or Housing of Odor-Producing Processes

A number of unit processes in wastewater treatment plants are usually housed in buildings. Housing is normally done for climate protection of the equipment and for ease of operation and maintenance rather than for containing odors. Mechanical dewatering equipment is almost always housed, regardless of plant size. Preliminary treatment works, such as screens and grit chambers, are often housed for medium to large plants, but are normally sited outdoors for small plants.

Larger plants may employ large buildings to house sludge handling processes, such as thickeners, conditioning processes and elutriation tanks, and dewatering equipment. In severe northern climates, many processes are either housed or covered to protect against freezing. These include preliminary treatment works, primary clarifiers, fixed film processes such as trickling filters and rotating biological contactors, secondary clarifiers, and sludge handling processes. Where weather conditions or site constraints are severe, aeration basins may also be covered. For cases in which normally unhoused processes are enclosed for protection from extreme climate conditions, special precautions must be taken to ensure adequate ventilation to control humidity and resulting corrosion potential within the enclosed space.

The preceding discussion describes general design practices for housing or covering unit processes in wastewater treatment plants to protect equipment from climatic exposures and to facilitate operation and maintenance. It is often difficult during design to predict the source and severity of odor generation and to design covers or enclosures accordingly. In addition, covering of odor-producing units often requires collection and treatment of exhaust air, the cost-effectiveness of which must be weighed against other odor control alternatives, such as air or oxygen injection or chemical addition.

Certain unit processes normally sited outdoors have a very high potential for odor generation. These include gravity sludge thickeners, solids separation devices for thermally conditioned sludges, sludge blending tanks, and septage receiving/holding tanks. Mechanical sludge thickeners, such as dissolved air flotation and centrifugation units are normally housed. It is also recommended that gravity sludge thickeners and septage holding tanks sited outdoors be covered, and that exhaust air be continuously removed and treated to prevent escape of odors to surrounding neighborhoods (4).

Scum pits and holding bins for screenings, grit, and septic solids are major odor sources, and should always be covered to prevent odor escape.

6.3.4 Ventilation

Adequate ventilation is required for any enclosed areas in wastewater treatment plants where plant employees may be present. The need for ventilation is particularly critical where H_2S is present, not only for odor control, but for prevention of potentially hazardous working conditions and control of relative humidity. Recommended ventilation rates for sludge handling buildings range from 6 to 12 air changes per hour with a relative humidity of not greater than 60 percent. In areas where climate permits, sludge handling buildings with seasonal enclosures are recommended; thus maximum benefit may be obtained from natural ventilation for at least part of the year.

6.3.5 Construction Materials

Many construction materials can physically absorb odors by chemisorption, condensation, or chromosorption. The degree to which materials may absorb odors is dependent upon porosity, coarseness, color, and composition. For buildings housing odor-producing processes, such as raw wastewater screens and grit chambers or sludge dewatering equipment, particular care should be taken to select nonabsorbent construction materials. Dark, rough bricks, gray, porous concrete, and dark plastics readily absorb odorous compounds and then emit these odors for long times thereafter. To minimize or prevent odor absorption, the exposed surfaces should be dense, smooth, light in color, chemically stable or inert, and poor conductors of heat. Materials, such as glazed vitrified clay or glazed ceramic, make excellent nonabsorbing surfaces.

6.3.6 Maintenance Provisions

Poor housekeeping practices are a secondary, but important potential cause of odor generation at many wastewater treatment plants. For this reason, plant designs must include adequate provision for cleaning and flushing of channels, scum pits, holding tanks, screens, grit conveyors, and other unit processes in which solids and slimes may accumulate and generate odors.

Hose bibs for pressurized process water should be located throughout the plant such that less than 30 m (100 ft) of hose is required to reach all unit processes. All components should be readily accessible to facilitate cleaning. Drainage systems should be properly located and sized to allow easy removal of flushing water. Floors should be sloped for easy collection and removal of flushing water. These requirements are particularly critical for sludge handling buildings.

All tanks and process equipment should be designed to allow complete dewatering and subsequent access by plant personnel for cleaning and flushing.

6.4 Design Procedures for Specific Odor-Producing Unit Processes

Table 6-2 presents a matrix of potential odor-producing unit processes in wastewater treatment plants and recommended methods for odor control. Several of these processes, such as suspended growth systems, secondary clarification, tertiary filtration, and disinfection, are not normally major sources of odors. Others, however, discussed in greater detail below, often generate odors.

6.4.1 Headworks

Included under headworks are flow equalization basins, screens, grit chambers, and preaeration basins. The first consideration is whether or not the incoming wastewater is septic. If this is the case, it is likely to be more cost-effective to control sulfides upstream of the headworks than to collect and treat the odorous gases released during preliminary treatment. Upstream treatment techniques that have been successfully used to control odors at the headworks include air injection, oxygen injection, and addition of chemicals, such as hydrogen peroxide or chlorine. These alternatives are discussed in detail in Chapters 3 and 4. If the wastewater entering the headworks contains no sulfide, potential still exists for odor release due to the presence of other volatile odorants in the raw wastewater.

Flow equalization basins are likely candidates for potential development of septic conditions and subsequent odor generation. Design of equalization basins should incorporate an aeration system to maintain aerobic conditions and to keep solids in suspension. The minimum air supply rate as recommended by the "Ten State Standards" is 8.4 m^3/min/1,000 m^3 (1.25 cfm/1,000 gallon) of storage capacity (3).

Screening and grit removal processes are significant sources of odors due to the potential for accumulation of putrescible organics and other debris. These materials must be removed on a daily basis, and the units must be cleaned and flushed regularly to prevent odors. Grit and screening transfer systems such as conveyors should be designed to prevent spillage and to convey to the point of ultimate disposal with minimum detention time. Conveyor drip pans should be sloped to drain at one end, and should be accessible for cleaning and flushing. Accumulated screenings and grit should be kept in closed containers before disposal. An adequate pressurized water supply should be provided nearby to allow for cleaning and flushing of all preliminary treatment units (4).

Preaeration is sometimes used to prevent septicity, to improve grease removal, and to promote flocculation of wastewater solids prior to primary clarification. Detention times vary from 15 to 60 minutes. The value of preaeration in preventing septicity is questionable, due to the short detention times as previously discussed. One study showed the effects of 30 minute preaeration on ORP to be only temporary. Although the ORP of the raw wastewater was

Table 6-2. Matrix of Potential Odor-Producing Processes and Recommended Odor Control Methods

Unit Process	Recommended Control Methods: Chemical, Air or O_2 Addition Upstream of Plant	Aeration	Chemical Addition	Covering With Collection and Treatment of Air	Improved Hydraulics to Avoid Turbulence	Improved O & M
Flow Equalization	X	X	X			
Preliminary Treatment						
Screening	X			X		X
Grit removal	X			X		X
Preaeration	X			X		
Liquid Stream Treatment						
Primary clarification	X		X	X		X
Suspended growth systems						X
Fixed film systems			X	X		X
Phys/chem systems			X	X	X	X
Secondary clarification						X
Tertiary filtration						
Disinfection						
Sidestream returns		X	X		X	
Sludge Stream Treatment						
Gravity thickening			X	X		
DAF thickening						
Blending and storage			X	X	X	
Aerobic digestion						X
Anaerobic digestion						X
Chemical stabilization				X		
Thermal conditioning						
Mechanical dewatering			X	X		
Drying beds			X			X
Composting						X
Septage receiving/holding		X	X	X	X	

increased from 10 mV to 170 mV, ORP was reduced to 35 mV after primary sedimentation. Another study, however, concluded that a 60-minute preaeration time was sufficient to achieve an ORP that did not decline during subsequent sedimentation (4). Preaeration of sulfide-bearing wastewaters is not recommended, as the turbulence induced by aeration will release H_2S and other odorous gases to the atmosphere.

6.4.2 Liquid Stream Processes

This discussion focuses on liquid stream treatment processes which have relatively high potential for odor generation or release. Included in this category are primary clarification, physical-chemical systems, and sidestream returns. Although not considered to be normally significant sources of odors, activated sludge and trickling filter systems are also discussed, since improper design of these processes can result in odor generation.

Primary clarifiers are potential odor sources due to the relatively long liquid residence times, particularly when actual flows are considerably less than design flows, which can lead to septic conditions. The problem may be further compounded by the presence of septic conditions in the raw wastewater. Unfortunately, since clarifiers are normally sized to accommodate peak design flows, little can be done during design of the clarifier to eliminate this problem. Preaeration for a minimum 60-minute detention time can be effective in preventing septicity in the primary clarifier, and should be considered during design if clarifier detention times will be excessive at less than design flows, and if the raw wastewater contains no sulfide. If sulfide is present, preaeration will release sulfides and create severe odor problems. Sulfide release from accumulated sludge and scum removal was discussed in Section 6.2.1.

Design of covered primary clarifiers, with collection and treatment of exhaust air, would only be considered during design if sulfides are known to be present in the raw wastewater and if upstream treatment with air, oxygen or chemicals is not practical. It is recommended that generation of odors in primary clarifiers resulting from excessive detention times at low actual to design flows be controlled by temporary solutions, such as prechlorination or addition of hydrogen peroxide.

Odors from activated sludge basins generally result from organic overload conditions (inadequate DO) or from poor mixing characteristics. For diffused air systems, air requirements to ensure good mixing are 20-30 m^3/min/1,000 m^3. Power requirements to ensure complete mixing with mechanical aerators are 13 to 26 kW/1,000 m^3 (0.5 to 1.0 hP/1,000 ft^3) (2). These will vary with actual basin and aerator characteristics.

Trickling filters may produce odors due to organic overload conditions, inadequate wetting rates, ponding conditions, or improper ventilation. Design organic loading rates, which are dependent on filter type (low, intermediate, or high rate) and media type (rock or synthetic), are 0.08 to 4.8 kg BOD/m^3/d (5 to 300 lb BOD/1,000 ft^3). For plastic media trickling filters, minimum wetting rates are 5.7 to 28.5 l/min/m^2 (0.14 to 0.7 gpm/ft^2), depending on the particular media employed (5). Ventilation is important in trickling filter design to prevent odor generation. Normally ventilation occurs naturally due to the temperature differential of the wastewater and the ambient air. If air temperature is higher than the wastewater temperature, the flow is downward, and vice versa. General design recommendations for ventilation of trickling filters are as follows (2)(3)(5):

1. Underdrains and collection channels should be designed to flow no more than half full at peak design flows.
2. Underdrains should have a minimum slope of 1 percent, and effluent channels should be designed for velocities of 0.6 m/s (2.0 ft/sec) at average design flow.
3. Ventilating manholes with open-grate covers should be installed at both ends of the central collection channel.
4. Large diameter filters should be designed to provide branch collection channels with ventilating manholes or vent stacks at the filter periphery.
5. Open area of the slots in the top of underdrain blocks should be a minimum of 15 percent of the filter area.
6. One square meter of gross area of open grating in ventilating manholes or vent stacks should be provided for each 250 square meters of filter area.
7. For plastic media trickling filters manufacturers often recommend 0.1 m^2 (1 ft^2) of ventilating area for each 3 to 5 m (10 to 15 ft) of filter periphery.
8. Forced-air ventilation should be employed for extremely deep or heavily loaded trickling filters.

Physical-chemical treatment systems are likely candidates for odor generation, since lack of aeration results in a low ORP, and use of chemicals such as lime raises the pH and can result in generation of ammonia odors. Conversely, chemicals such as ferric chloride can reduce the pH, which favors release of H_2S. Since turbulence will accelerate the release of odorous gases, turbulence should be minimized where possible. Mixing tanks should be covered to avoid escape of odors. Depending on the chemical treatment processes employed, chemical addition might be used for odor control. Unfortunately, lack of data from the limited number of operating physical-chemical systems makes firm design recommendations difficult. It should be noted, however, that common design practice has been to enclose the major elements of the physical-chemical treatment plant, which has resulted in severe corrosion problems in some cases.

Sidestream returns from sludge handling operations such as thickening, digestion, and thermal conditioning are often significant sources of odors. Odor release from anaerobic sidestreams usually results from turbulence created when the sidestream enters the main plant flow. For this reason, sidestreams should always be returned below the surface of the liquid. Adverse effects on primary clarification, such as septicity and odors, can be avoided by returning the sidestream to the biological process. Mixing of the sidestream with waste activated sludge prior to return to upstream processes has been used to promote adsorption of odors by the activated sludge particles. Chlorination of sidestreams at dosages of 100 to 300 mg/l has also been employed for odor control, as has aeration in an enclosed conduit (4)(6). Separate treatment facilities have been designed for high strength sidestreams such as those from thermal conditioning processes, although such systems are generally used primarily to reduce BOD and SS loadings to upstream processes.

6.4.3 Sludge Stream Processes

Sludge handling processes with high potential for odor generation include gravity thickening, sludge blending and storage, thermal conditioning, chemical stabilization, mechanical dewatering, composting, and septage receiving and holding.

Gravity thickeners are common sources of odors because primary or primary/waste activated sludges generally have low ORP values, and may become septic during detention in the thickener. Odor problems result from feeding of thick, aging sludges to the thickener, without providing sufficient aerobic dilution water to "freshen" the sludge. Increasing rates of sludge withdrawal can sometimes avoid septicity, although intermittent odor problems are still likely. It is, therefore, recommended that gravity thickeners be covered to contain odors. Covers may be of the flat, low-profile type, or self-supporting aluminum or fiberglass domes. Since flat covers do not allow ready access for observation, equipment maintenance, or cleaning, dome-type covers are usually recommended for gravity thickener applications. To ensure personnel safety, the enclosed space must be adequately ventilated. Ten to twenty air changes per hour are typical for such spaces.

For gravity thickeners, designs with covers, and forced-air ventilation, the exhaust air must undergo suitable treatment prior to discharge to the atmosphere (see Chapter 4). An exception to the recommendation of covering gravity thickeners is when the incoming sludge solids are fresh and aerobic, as might occur from extended aeration plants (>20 day SRT) with no primary clarification (7).

Sludge blending and storage tanks also may be sites of significant odor generation, particularly when raw primary sludge is being blended or stored with other plant sludges. For sludges that have undergone aerobic digestion or that are derived from extended aeration processes, odor generation is likely to occur only if the sludge is stored for sufficiently long periods (>2 hours) to allow development of septic conditions.

Sludge blending and storage tanks should normally be covered to contain odors. This is typically accomplished with a sealed, flat cover. Manways are provided for access to equipment. The most positive means of odor control from blending or storage tanks is direct combustion of the vapors at approximately 760°C (1,400°F). Because gas production is low and the enclosed air space is small with a flat cover, energy requirements are moderate and electrical heat can be used in combustion for simplicity (7). For flat, tight-fitting covers, ventilation rates need only be sufficient to maintain a slight, negative pressure under the cover. Recommended ventilation rates for such applications are four to six air changes per hour (8). Treatment of exhaust air is discussed in detail in Chapter 4.

Thermal sludge-conditioning systems are potentially major sources of objectionable and complex odors. The odorous gases produced are low molecular weight volatile substances that include aldehydes, ketones, organic acids and various sulfurous compounds. The odor levels produced depend on the total hydrocarbon content of the individual odor source. All sludge processing operations downstream of the reactor will produce odor because of the low vapor pressure of the volatile odorous compounds and the relatively high sludge temperatures. The most common sources of odor are: 1) vapors from treated sludge storage tanks; 2) decant tanks; 3) thickeners; 4) exhaust air from vacuum filter pumps; 5) exhaust air from other loaded or enclosed dewatering devices; and 6) vapors released from further transport and treatment of decant liquors.

Of the preceding, the odor levels are most severe from decant tanks or thickeners immediately following the reactor. Odorous air from sludge decant tanks, thickeners, separate strong liquor pretreatment systems, sludge loading or transfer hoppers, and vacuum dewatering equipment should be collected and treated before being released to the atmosphere. Vacuum filter pump exhaust must also be collected for treatment to remove odors.

The more commonly used alternatives to control odors in the exhaust air from thermal sludge treatment systems include wet scrubbing, combustion, and activated carbon adsorption. Masking, dilution, and evaporation control also have been used, although their effectiveness is limited. For the air streams with high hydrocarbon content, such as those produced by sludge thickeners and decant tanks, the most effective odor control system is incineration or wet scrubbing followed by incineration. These systems will reduce odor levels to a range of 880 to 3,530 odor units/m^3 (25 to 100 odor units/scf). Wet scrubbers can be employed using plant effluent as the scrubber liquid at rates of 2.7 to 4.0 l/min per m^3/min of air flow (20 to 30 gpm/1,000 scfm). Incineration can be either by direct flame at 815°C (1,500°F) or by catalytic combustion at 425°C (800°F).

Wet scrubbing plus activated carbon adsorption can also be used for odor removal from the high hydrocarbon content gas streams. Scrubber liquids commonly are solutions of potassium permanganate, sodium hydroxide, or sodium hypochlorite. Activated carbon adsorption normally uses a mutiple bed unit sized to minimize regeneration requirements. Regeneration of the carbon for such applications is typically accomplished by steam stripping. For a 28-m^3/min (1,000 cfm) air flow, a typical carbon system would use dual beds, each containing 820 kg (1,800 lb) of carbon and an adsorption cycle of 24 hours. After 24 hours, the second bed would be placed into operation, and the first bed would be regenerated for 1 hour using steam (9). The actual adsorption cycle is a function of hydrocarbon content and should be determined under actual plant conditions.

The third alternative for treating the high hydrocarbon gas streams is use of multiple wet scrubbers. One of the most effective multiple scrubber options is to use three stages, the first employing plant effluent as the scrubber liquid, the second using a 5-percent sodium hydroxide solution, and the third using a 3-percent potassium permanganate solution.

Other techniques for odor control from thermal sludge treatment systems include the use of a nitrifying trickling filter as a scrubber, discharge of the gases to aeration basins, and discharge of gases to aerated lagoons (9).

The most effective method of odor control from thermal sludge conditioning processes depends on the specific chemical composition of the odorous gas, odor strength, volume of air to be treated and odor reduction required. The degree of odor control depends on site characteristics, including topography, climate, prevailing wind direction and proximity to populated areas.

Based on a survey of 28 operating facilities, the most effective methods of odor control were found to be high temperature incineration, activated carbon adsorption, and wet chemical scrubbing.

The costs for treating odorous gases from thermal conditioning processes are 5 to 10 percent of the total cost of thermal treatment. For systems treating 3 to 30 m^3/min (106 to 1,060 cfm) of odorous air, wet chemical scrubbing was found to be the most cost-effective method, followed by incineration and carbon adsorption. Detailed costs are presented in Chapter 4 (9).

Chemical stabilization processes, particularly lime stabilization, are potential sources of odors. In lime stabilization systems, pH of the sludge is elevated to 12.0 for 2 hours in order to prevent biological decomposition and generation of noxious odors. However, elevation to pH values above 9.5 favors release of ammonia to the atmosphere. In addition, turbulence caused by mixing of lime and sludge accelerates the rate of ammonia release. For this reason, such mixing tanks should be covered to minimize the escape of odors.

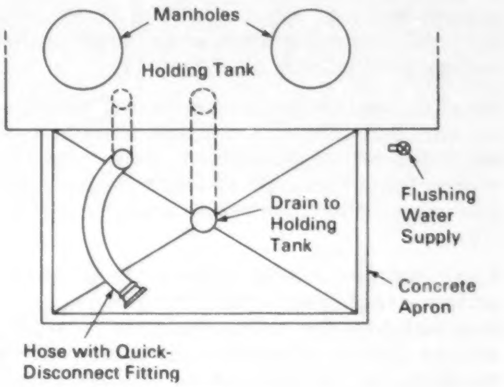
Sludge composting processes are likely sources of odors, primarily due to the requirements for handling and transfer of materials. These processes involve mixing of raw sludge with bulking agents, transfer of these materials to piles or windrows, periodic turning of windrows, and removal and storage of compost. Because non-mechanical sludge composting systems are land-intensive, relatively remote sites are often selected with adequate buffer zones to allow dispersion of any odors that might be generated. Static pile composting systems employ small blowers to "pull" air through the pile for aeration; the exhaust air is passed through a small pile of finished compost for adsorption and removal of odors. Finished compost has been found to be effective for this purpose. Composting systems can be enclosed in buildings, and odors controlled through proper ventilation and treatment of the exhaust air. In most cases, however, operations are conducted outdoors, making site selection the key design criterion for odor control.

Septage is a putrescible organic material with a highly objectionable and persistent odor. Special consideration must be given to control odors from septage receiving stations and holding tanks at wastewater treatment facilities. Receiving stations must be designed to minimize spills and turbulence during discharge of septage from hauling vehicles. A hose from the holding tank equipped with a quick-disconnect, watertight fitting should be provided for direct transfer of vehicle contents to the holding tank or to pretreatment facilities, such as bar screens.

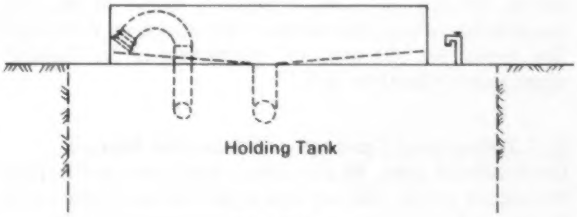
To allow for discharge of septage from vehicles not equipped with compatible hoses or fittings, a hopper should be provided which drains to the holding tank (10). Such a design is shown in Figure 6-1. A pressurized water supply must be provided for flushing any spills and for cleaning the facilities. Receiving tanks should be totally enclosed to prevent escape of odors.

Although aeration times required for stabilization and odor control are excessively long for application to holding tanks, aeration can release odorous gases so as to minimize odor release in downstream processes. Forced-air ventilation should be provided to exhaust the odorous gases, which must be treated before discharge to the atmosphere. In Europe, the most common technique is to use wet scrubbers, with sodium hypochlorite as the scrubbing liquid. Use of soil or compost filters, discharge of the gas to activated sludge basins, and combustion have also proven successful. However, use of activated carbon for odor removal has not been very successful, due to incomplete odor removal and operational expenses of carbon replacement (10). Ozone has also been used for treatment of odors from exhaust gases emanating from septage-receiving tanks (11).

Figure 6-1. Typical design of a septage receiving station.



Plan



Elevation

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6.5 General Design Considerations for Avoiding Corrosion

Because of the characteristics of wastewater and the unit processes employed in wastewater treatment plants, corrosion is always a potential problem. Repair or replacement of components due to corrosion can be costly, and it is therefore important to prevent or minimize potential for corrosion during design. Additional costs incurred for specification of materials with high degrees of corrosion resistance and for designs that minimize corrosion are generally insignificant compared to the costs of repair and replacement of corroded components over the typical 20-year design lifetime of the plant. The following discussion presents general design considerations for avoiding corrosion in wastewater treatment facilities.

6.5.1 Avoiding Moisture Retention and Ponding

Design of channels, angles, and structural beams should be such that possible catchment areas for liquids and moisture are avoided. In the use of structural steel, areas that are difficult to clean and maintain, such as back-to-back angles or structural sections with inaccessible areas, should be avoided, as should flat or dished sections that can collect or retain moisture. Corners should be rounded where possible to prevent accumulation of dirt and moisture which may act as an electrolyte to induce corrosion. If it is not possible to avoid catchment areas for accumulation of liquid, drainage holes should be provided. These must be sized and sited carefully, and maintained free from blockages (12).

For purposes of minimizing moisture and dirt accumulation, butt-welded joints are preferable to lap-welded joints. If lap-welded joints must be used, exposed edges should be treated in such a way as to prevent retention of moisture and dirt in the crevices (12).

Steel storage containers and tanks should be supported on legs to allow free circulation of air over the tank surface to prevent condensation. Insulation can also be used to prevent condensation. Condensed moisture can be retained on sheltered horizontal surfaces, such as building eaves or undersides of tanks. Where possible, breathing holes should be provided to allow circulation of air and evaporation of the moisture. Control of condensation is further discussed in Section 6.5.3.

6.5.2 Avoiding Contact of Dissimilar Metals

Bimetallic or galvanic corrosion, described in Chapter 4, occurs when two or more dissimilar metals are immersed in, or are conveying, an electrolyte; examples are: use of bronze or brass valves with iron pipe, use of steel rivets to fasten aluminum sheets (or vice versa), and use of steel and brass or copper pipe in the same system. Contact of dissimilar metals should always be avoided, where, for example, steel rivets are used for fastening aluminum sheets, corrosion of the aluminum sheets can result in loosening of the rivets, slipping of the sheets, and potential structural damage. Such corrosion can be prevented by using an insulating, non-hardening compound in the areas where the two metals come in contact (12).

6.5.3 Ventilation and Heating for Condensation Control

Ventilation and heating of enclosed spaces effectively controls condensation and corrosion. Such control is particularly important where enclosed areas are exposed to open water surfaces such as in covered grit chambers and screens, wet wells of pump stations, covered sludge thickeners, sludge handling buildings, other equipment areas, and pipe galleries.

As an example, at Yellow Springs, Ohio, freezing problems occurred in the grit removal mechanisms in an uncovered, aerated grit chamber. The units were covered in an attempt to control ice formation. Although freezing problems were controlled, failure to provide adequate ventilation resulted in accumulation of condensates on mechanical components, which led to rapid and severe corrosion of metallic parts.

Enclosed spaces should be ventilated by forcing fresh air into the enclosure in order to displace air containing high levels of moisture. This has proved successful at many locations, including Detroit, Michigan; Massilon, Ohio; Winona, Minnesota; Circleville, Ohio; and Pontiac, Michigan (12). Areas that were ventilated in these cases included wet wells, chambers and pump rooms, pipe tunnels, screen and comminutor rooms, and the space between the roof and bottom plate of a floating cover digester (12). "Ten State Standards" recommends ventilation of wet wells at a rate of 12 air changes per hour, and ventilation of dry wells at 6 air changes per hour, if continuous. If intermittent, the recommended rate for both wet and dry wells is a minimum of 30 air changes per hour (3). Separate ventilation should be employed for wet and dry wells. If intermittent, ventilation equipment should be interlocked with the lighting system. Effective ventilation is accomplished by forcing fresh air into the bottom of the structure and exhausting it through the roof (13).

The design of heating, ventilating, and air conditioning systems must include consideration of both minimum heating and ventilation requirements and humidity control. Humidity control is especially important for enclosed areas that house open tanks and for subsurface structures. Unnecessary heating of the air should be avoided, since warm air promotes condensation on tanks, pipes and basins that contain cooler wastewater. The basic heat requirement is for freeze protection, with recommended temperatures of 4°C (40°F). Minimum ventilation requirements are specified by Federal, state, and local building codes for protection of health and safety. Ventilation rates range from 2 to 30 air changes per hour, depending on the occupancy classification, type of equipment or process that is housed, and the potential for generation of objectionable or hazardous air.

6.6 Paint and Coatings

The adequate protection of steel and concrete in wastewater treatment plants is important for minimizing maintenance and/or replacement of corroded components. As a result, it is critical that construction specifications contain provisions that will assure proper techniques are used for surface preparations, and that the appropriate primers, paints and coatings are specified for the type of environmental conditions to which these materials are subjected. Further, the paints and coatings must be properly applied to ensure a long life with minimal maintenance.

6.6.1 Types of Coatings

Corrosion of steel and concrete can be abated or prevented by coating them with materials which have greater resistance to corrosion. Two generic coatings are used: physical barrier coatings, which provide a barrier between the material to be protected and the environment, and sacrificial coatings such as zinc and cadmium, which corrode preferentially and save the primary base metal from attack. Coatings are further subdivided into metallic coatings, non-metallic organic coatings, and chemical conversion coatings.

6.6.1.1 Metallic Coatings

Zinc and cadmium coatings have a higher electromotive force than steel, and can be used to cathodically or galvanically protect iron and steel. Here, the coatings are corroded preferentially, preventing attack of the primary metal.

Application of zinc coatings (galvanizing) is normally accomplished by dipping the component in a molten zinc bath. It has been found that the effective service life of a zinc coating varies directly with the thickness of the coating. Service life of a galvanized coating also varies with the severity of the exposure (12).

Nickel coatings do not provide sacrificial protection. Rather, they must provide an impervious, non-porous physical barrier to prevent attack of the primary metal. Electroplated nickel coatings vary in thickness from 0.5 to 10 mils depending on the exposure. Such coatings are typically applied over a very thin layer of copper to improve adhesion. Nickel can also be applied by electroless plating and by cladding. Other electroplates, such as chromium and silver, are also useful for some corrosive environments.

Metallic coatings such as aluminum, tin, lead, monel, and stainless steel are often used for corrosion protection. Hot-dipped aluminum coatings have been found to be useful in high temperature, corrosive environments, since they have a high resistance to corrosive condensates which form when the heated component cools down.

Inorganic zinc coatings have been developed which consist of metallic zinc particles in a vehicle such as sodium silicate. A curing agent or hardener is employed to complete the chemical reaction during formation of the coating. Such coatings bond tightly to the base metal surface, and protection is afforded by the preferential corrosion of the zinc and by the production of stable, insoluble corrosion products such as hydroxides, oxides, and carbonates (12).

6.6.1.2 Non-Metallic Organic Coatings

Organic coatings provide a protective barrier between the surface to be protected and the environment. Organic vehicles such as thinners, drying oils, and resins are used in such formulations. Synthetic resins are commonly used to enhance the ability of the coating to resist acids and alkalies (12). Vinyl resins provide impervious surfaces that resist penetration by water. Epoxy resins are becoming quite popular, as they show good chemical resistance and excellent surface adhesion. Silicone resins are used for high temperature service.

6.6.1.3 Chemical Conversion Coatings

Chemical conversion coatings are so named because of chemical reactions that occur between the coating and the base metal; the coating becomes an integral part of the original surface. Two common chemical conversion coatings are phosphate coatings and controlled oxidation coatings.

Phosphate coatings are produced by the chemical reaction of the primary metal with a phosphoric acid solution containing zinc, iron, or manganese along with iron phosphates. A crystalline, non-metallic layer is formed on the surface of the metal. Such coatings have found greatest use as a base to provide better adhesion of paints.

Controlled oxidation coatings are formed by exposing metal components to hot oxidizing liquids or gases. This results in the formation of a thin (0.02 to 0.2 mil) black oxide coating that can provide protection against corrosion or serve as a base for painting.

6.6.2 Surface Preparation

6.6.2.1 Steel Surfaces

Proper preparation of steel surfaces is important for assuring good adhesion between the coating and the surface. Sand or shot blasting are methods commonly used for thorough cleaning of the steel, both in fabrication shops and in the field. Other cleaning methods, such as solvent cleaning and hand tool cleaning, are also discussed.

The Steel Structure Painting Council (SSPC) has issued the following specifications for various degrees of sand or shot-blasted preparation of metal surfaces (12)(14):

1. SSPC-SP5, white metal—Blast cleaning for complete removal of all visible rust, mill scale, paint, corrosion products, and foreign materials.
2. SSPC-SP10, near white metal—Blast cleaning for 95 percent removal of visible residues; minimum required for immersion service.
3. SSPC-SP6, commercial—Blast cleaning for 67 percent removal of visible residues.
4. SSPC-SP7, brush off—Blast cleaning for removal of all but tightly adhering residues of rust, mill scale, and coatings.

Other surface preparation specifications are:

1. SSPC-SP1, solvent cleaning—Removal of all oil, grease, dirt, salts and contaminants by cleaning with solvents, vapors, alkali emulsions, or steam.
2. SSPC-SP2, hand tool cleaning—Removal of loose rust, mill scale, and paint by hand chipping, scraping, sanding, or wire brushing.
3. SSPC-SP3, power tool cleaning—Removal of loose rust, mill scale, and paint by power tool chipping, sanding, wire brushing, or grinding.
4. SSPC-SP8, pickling—Complete removal of rust and mill scale by acid pickling. Iron phosphate coating is produced which improves paint adhesion.

Table 6-3 summarizes the recommended preparation technique for steel surfaces depending on the conditions of exposure (14). For the most severe exposures, blasting to near white or white metal provides the best conditions for good adhesion of the paints or coatings, while less severe exposures call for less extensive surface preparation alternatives.

Table 6-3. Surface Preparations and Coatings for Various Environmental Exposures (14)

Exposure	Surface Preparation for Steel	Minimum Coating Film Thickness	Recommended Coating
		mils	
Submerged	Minimum of near-white metal blast SSPC-SP10*	7	Epoxies Vinyls Coal-tar epoxies
Moist Atmosphere	Commercial blast SSPC-SP6	5	Epoxies Vinyls Chlorinated rubber Coal-tar epoxies
Outside Weather	Commercial blast SSPC-SP6	5	Steel: Alkyds Epoxies Concrete: Chlorinated rubber High-build epoxy
Inside, Dry Atmosphere	Hand tool cleaning SSPC-SP2	3-5	Steel: Alkyds Concrete: Chlorinated rubber Epoxy

*Steel Structure Painting Council Specifications

6.6.2.2 Concrete Surfaces

Concrete surfaces must be prepared for paints and coatings by thoroughly removing all grease and oils, dirt, scale, and loose and foreign materials to provide good adhesion of the coating with the concrete. Vinyls and chlorinated rubber coatings require good surface preparation since they have relatively weak bonding properties. One of the best preparations for concrete floors is acid etching by swabbing with a solution of muriatic acid followed by thorough rinsing. For concrete walls, a zinc sulfate solution is often used. The zinc sulfate combines with calcium hydroxide to form zinc hydrate [$Zn(OH)_2$] and calcium sulfate ($CaSO_4$), both of which are paint pigments. A 2-percent zinc chloride, 3-percent phosphoric acid solution is also effective for preparing concrete surfaces. For concrete surfaces that are greasy or oily, contaminants may be removed with trisodium phosphate, solvents, or caustic lye (12).

6.6.2.3 Galvanized Iron Surfaces

Galvanized iron surfaces can be further protected by application of paints. If painting is desired, the surface must be adequately prepared to permit bonding of the paint. Two methods are commonly used:

1. Weathering—Weathering is a natural phenomenon which produces a roughened surface due to the oxidation of zinc to zinc oxide. This changes the surface from a shiny finish to a dull gray surface to which the paint will bond.
2. Application of primers—When it is not desirable or practical to wait for natural weathering to occur, primers can be used to improve the adhesion of paints. These include a vinyl wash coat (phosphoric acid solution) followed by a zinc dust primer, acetic acid, and zinc dust—zinc oxide primers.

6.6.3 Selection of Primers, Coatings and Paints

6.6.3.1 Primers for Steel

Primers can be divided into two types: inhibitive primers, such as zinc chromate and red lead; and barrier primers, which protect the surface by providing a mechanical barrier to corrosive agents.

Red lead primers are alkaline, and can neutralize acidic agents that may penetrate the film. Red lead pigments react with oils to form dense, tough films with low permeability.

Zinc chromate is an effective inhibiting agent due to the slow release of chromate ions. Zinc chromate primers are not recommended for acidic environments or for immersion service.

Inorganic zinc primers are very effective due to their ability to provide sacrificial protection, as well as to develop a resistant coating with time. These are excellent primers for immersion service. In addition, special inorganic zinc preconstruction primers are available which need not be removed before welding. Such primers offer excellent protection for steel between arrival at the job site and application of topcoats (15).

Wash coat primers are actually pretreatments, as their purpose is to improve adhesion of subsequent paints or coatings.

6.6.3.2 Paints and Coatings

a. Coal-Tar Epoxy
Coal-tar epoxy coatings are often specified for submerged surfaces such as clarifiers, digesters, and process tanks. They do not crack when exposed to sunlight, are not softened by oils and fats, and demonstrate good adhesive properties and abrasion resistance. Coal-tar epoxy can be applied to both steel and concrete surfaces.

Coal-tar epoxies employ a two-component, cold curing system using either amine or polyamide curing agents. Many specifications for field applied coal-tar epoxy call for two coats applied over near-white blasted steel, or two coats over concrete, the first applied at reduced viscosity to allow penetration and improved bonding.

A disadvantage of coal-tar epoxy coatings is that, in order to assure good bonding, the top coat must be applied soon after the previous coat, generally within 24 to 72 hours, depending on temperature and formulation (14). Table 6-3 shows some recommended applications and film thicknesses for coal tar epoxies.

b. Epoxies
Epoxies have a multitude of applications as coatings in wastewater treatment plants. They are extremely effective for submerged service applications on steel or concrete. Epoxies are durable, adhesive, and provide excellent resistance to acids, alkalies, solvents, abrasion, and impacts. They are thermosetting, and can be cured by heat or by internal polymerization using organic amines as curing agents. Polyamide-cured epoxies do not provide the solvent and chemical resistance of amine-cured epoxies, but have higher solids contents, better adhesion, moisture tolerance, and flexibility. Topcoats must generally be applied within 72 hours. When exposed to sunlight, epoxy coatings chalk and lose their gloss, although this does not affect the integrity of the film (14). Epoxy coatings have become quite popular, and seem to be replacing coal tar epoxies for wastewater treatment plant applications. Table 6-3 shows some recommended applications and film thicknesses for epoxy coatings.

c. Vinyl Coatings
Vinyl-resin coatings can be used for submerged service and moist atmosphere exposures of concrete and steel. Because vinyl-resin coatings are high in viscosity and low in solids content, films are thin, requiring three to four or more coats depending on the application. Vinyl coatings have very low permeabilities, and are resistant to oils and fats, alkalies, and many chemicals, although they can be attacked by acetic and other organic acids. Although they are excellent for submerged service applications, the thin film thickness requiring multiple coats has generally favored use of high-build epoxy coatings. For corrosive environments exposed to sunlight, vinyl coatings are often preferred since they do not chalk and fade rapidly. Applications and some recommended film thicknesses for vinyl coatings are summarized in Table 6-3.

d. Alkyd Resin Coatings
Alkyd resins form hard durable films with good resistance to dulling and fading in outdoor exposures. They are not suitable for submerged or moist atmosphere exposures or for coating of concrete. A common application of alkyd resins (see Table 6-3) is for protection against industrial exposures of interior and exterior metal surfaces.

e. Phenolic Resin Coatings
Phenolic resins, made from phenol and formaldehyde, are often used as primers. Although superior to alkyds for chemical resistance,they are inferior to vinyls, epoxies or chlorinated rubber coatings under severe conditions. Certain types of phenolic resins are suitable for submerged exposures. Because they form hard, insoluble films, adhesion of topcoats may be a problem (14).

f. Chlorinated Rubber Coatings
Chlorinated rubber compounds are easily applied, and can be used for both steel and concrete. Although chlorinated rubber has good resistance to H_2S gas

and moisture, it is not suitable for submerged exposures because of poor resistance to oil, grease and solvents.

g. Emulsion Coatings

Most emulsion-type coatings are made from acrylics, polyvinyl acetate, butadiene styrene, or combinations of the above. They are commonly used on concrete and concrete block walls. They are easily applied, do not have strong solvent odors, are not a fire hazard, and have good gloss and color retention. Cleanup is easily accomplished with water. They must be applied at temperatures in excess of 10°C (50°F).

6.7 Selection of Materials

A large number of materials, both metallic and non-metallic, are used in the construction of wastewater treatment plants. It is important during design to be cognizant of the potential corrosion problems that may occur, and to carefully select materials that have a high degree of corrosion resistance and require little maintenance. Greater capital investments for such materials are justified by the savings in maintenance and replacement costs over the lifetime of the plant. Materials typically used in wastewater treatment plants are discussed below relative to their durability and degrees of corrosion resistance.

6.7.1 Cast and Ductile Iron

Cast and ductile iron corrodes at about the same rate of steel. However, because of their greater thickness and the formation of a dense, tenacious oxide which retards further corrosion, they hold up well in some corrosive environments. Gray cast iron may be subject to graphitization when immersed in salt water or moist, sulfate-bearing soils. This involves dissolution of ferrite in the cast iron, leaving the graphite intact, but reducing the density and structural strength of the component. White cast iron is not subject to graphitization.

6.7.2 Low Alloy Steels

The composition of low alloy steels has no appreciable impact on corrosion resistance in submerged or buried conditions. However, for atmospheric exposures, addition of chromium, copper, or nickel in small amounts (0.1 to 1 percent) results in formation of a dense, adherent, protective film which, upon oxidation, reduces the rate of corrosion. The type of atmospheric exposure will affect the rate of corrosion.

6.7.3 Copper and Copper Alloys

Copper and copper alloys have low positions in the electromotive series, and as such demonstrate good resistance to corrosion. Copper exposed to the atmosphere develops a thin, green protective coating that is largely copper sulfate. Copper offers good resistance to dilute, non-oxidizing acids and salt solutions. However, copper and its alloys can be readily attacked by H_2S, which turns the surface black. Copper is also sensitive to corrosion by high velocity waters containing high DO levels. Copper is not resistant to oxidizing acids (nitric, hot sulfuric acid), ammonium hydroxide (plus oxygen), and oxidizing heavy metal salts [$FeCl_3$, $Fe_2(SO_4)_3$].

Copper alloys such as brasses and bronzes, generally offer good resistance to corrosion. Brass is a copper alloy containing 5 to 45 percent zinc; white bronze contains either tin (up to 12 percent), aluminum (up to 10 percent) or silicon (up to 4 percent). The major corrosion process in brass is dezincification, or loss of zinc from the alloy, resulting in increased porosity and loss of structural strength. Tin or arsenic may be added to inhibit dezincification.

Bronzes are not subject to corrosion processes which remove one element such as in dezincification, and are generally stronger and harder than brasses. Aluminum bronze is the most resistant of any bronze to attack by H_2S and acids. Silicon bronzes are also resistant to corrosive compounds, particularly hydrochloric and sulfuric acids, alkalies, and some organic compounds.

6.7.4 Stainless Steel

Stainless steels are metal alloys containing chromium (> 11.5 percent) and, for some types, nickel (6 to 22 percent). Stainless steels demonstrate excellent corrosion resistance, and have been used in wastewater treatment plants for many applications, including flow control gates, aeration piping, handrails, and gratings. There are three basic classes of stainless steels:

1. *Martensitic*—These alloys have chromium contents of 11.5 to 17 percent, and carefully controlled carbon content. They may be hardened by heat treatment to yield a martensite structure. Applications include steam turbine blades, tools, and cutlery.
2. *Ferretic*—These are low carbon alloys containing 17 to 27 percent Cr. They can be hardened to some degree by cold working. The crystal structure is ferretic, and demonstrates superior atmospheric corrosion resistance than martensitic alloys. Applications include autombile trim and components exposed to nitric acid.
3. *Austenitic*—These are low carbon alloys containing 16 to 26 percent chromium and 6 to 22 percent nickel, with austenitic crystal structures. Addition of nickel improves corrosion resistance, making high nickel austenitic alloys superior to other types of stainless steels.

Stainless steels show good resistance to inorganic and organic acids and alkalies. However, they are not resistant to halides (Br, Cl, F), seawater, and oxidizing chlorides. Some stainless steels are subject to pitting and intergranular corrosion. However, such problems can generally be overcome by proper alloy selection, heat treatment, and exclusion of certain chemicals (12).

6.7.5 Nickel and Nickel Alloys

Nickel and high nickel (> 50 percent) alloys are excellent materials for corrosion resistance. Nickel alloys are stronger and harder than copper or aluminum alloys and offer superior resistance to corrosion. The six main types of high nickel alloys are described below (12):

Group I, Nickel: 93.5-99.5 percent nickel

- Excellent mechanical properties
- High strength, malleable
- Resists hydrogen chloride, caustic soda, oxidation and scaling, and stress corrosion in atmospheric exposures.

Group II, Nickel-Copper: 63-70 percent nickel, 29-30 percent copper

- "Monel" type alloys
- More resistant than nickel under reducing conditions
- More resistant than copper under oxidizing conditions
- Not resistant to strong solutions of nitric or sulfurous acid, ferric chloride
- Often used as wire mesh for vacuum filter cloth support, applications involving high velocity seawater (pump shafts, impellers, piping)

Group III, Nickel-Silicon: 85 percent nickel, 10 percent silicon

- Tough, strong, extremely hard
- Excellent resistance to corrosion by hot or cold non-oxidizing acids
- Sometimes used for pump and valve parts

Group IV, Nickel-Chromium-Iron: 54-78.5 percent nickel, 12-18 percent chromium, 6-28 percent iron

- Excellent corrosion resistance at high temperatures
- Withstands repeated heating and cooling
- Tough, strong, hard

Group V, Nickel-Molybdenum-Iron: 55-62 percent nickel, 17-32 percent molybdenum, 6-22 percent iron

- Excellent resistance to hydrochloric acid
- Expensive
- Exceptional cases of corrosion resistance requirements only

Group VI, Nickel-Chromium-Molybdenum-Iron: 51-62 percent nickel, 15-22 percent chromium, 5-19 percent molybdenum, 6-8 percent iron

- High corrosion resistance to oxidizing acids
- High resistance to thermal shock
- Hard and difficult to work
- Used for pump and valve parts, nozzles, piping exposed to oxidizing agents

6.7.6 Silicon Cast Iron

Commercial grades of silicon cast iron alloys contain 14.5 percent silicon. Addition of silicon improves corrosion resistance to strong, non-oxidizing acids. Such alloys have been used for pipes to convey waste chemicals, centrifugal pumps, valves, chlorine ejectors, spray nozzles, and agitators (12).

6.7.7 Aluminum

Aluminum is widely used in wastewater treatment plants due to its light weight, strength, and resistance to corrosion. Aluminum is not affected by H_2S, methane, carbon dioxide, or sulfur dioxide. Formation of a stable oxide coating on the surface by atmospheric exposure or by anodizing provides excellent resistance to corrosion. It may be attacked by acids, salts, or aggressive waters.

Typical applications in wastewater treatment plants include gratings, deck plates, railings, doors, window frames, and ladders (12)(16). Since aluminum is high in the electromotive series, care must be exercised to prevent contact with iron, steel or other metals that can result in galvanic corrosion.

6.7.8 Plastics

Plastics include a broad range of synthetic materials, and are divided into two major categories. Thermoplastics such as polyvinyl chloride, polyethylene, and vinyl can be heated to a plastic state, molded, cooled, then reheated and remolded. Thermosetting plastics such as polyesters, epoxies, and phenolics once formed cannot be reheated to a plastic state due to chemical changes which occur from the application of heat and pressure during forming.

Plastics demonstrate excellent resistance to a broad spectrum of corrosive materials such as acids and oxidizing chemicals, including ferric chloride, ferric and ferrous sulfate, and chlorine. The main disadvantage of plastics is their loss of strength at high temperatures. Thermoplastics are not normally used at temperatures above 65°C (150°F), while certain thermosetting plastics can be used at temperatures of up to 150°C (300°F). Plastics have high coefficients of expansion, are lower in strength than metals, and are relatively costly.

The range of applications for plastic in wastewater treatment plants is continually expanding. Plastics are used for pump impellers and casings, structural members, weirs, flumes, fans, fasteners, and laboratory equipment.

6.7.9 Elastomers

Elastomers include natural rubber, neoprene, butyl, isoprene, and others. The primary use of elastomers for wastewater applications is for sealants and gaskets. Neoprene has good resistance to oils and greases and oxidation, and is commonly used for such applications.

6.7.10 Ceramics, Glass, and Vitrified Clay

These materials are virtually immune to corrosion due to their inert, impervious surfaces. The major disadvantage with these materials is their brittleness.

6.7.11 Concrete

Portland cement concrete is the most widely used construction material in wastewater collection and treatment systems. In general, concrete is economical and provides excellent resistance to corrosion under both atmospheric and submerged exposures.

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National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650

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